The Development of a Bridge Management System Involving Standardised Scour Inspection Procedures and Flood Forecasting

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For Keni

2008-2020

"Simple solutions solve serious problems…"

I. K.

Table of Chapters

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Declaration

I hereby confirm that the body of work described within this thesis for the degree of Doctor of Philosophy, is my own research work, and has not been submitted for any other degree, either at University College Cork or elsewhere.

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Executive summary

Agencies charged with managing infrastructure face a difficult and very responsible task in managing their bridge stock. Historically, bridge management relied upon paper-based processing of information and the knowledge of engineering staff. However, if the knowledge gathered is not passed on to new staff, corporate memory loss occurs. Thus, bridge managers are increasingly using computer-based infrastructure management systems, e.g. Bridge Management Systems (BMS) in order to collect and store all information relating to bridges and to support their decision-making processes. A BMS is a means for managing bridge infrastructure during its lifetime (i.e. during design, construction, operation and maintenance of the bridges). A BMS comprises of collection of inventory data, bridge inspections (a fundamental of BMS), maintenance/repairs or replacement and allocation of funds. As the funds allocated to managing bridges are often significantly lower than for other infrastructure, for example annual allocations for road resurfacing, the requirement for an effective BMS becomes even more important. As funds are limited, prioritisation of bridge maintenance and maintenance is required. The outcome of most of bridge inspections is a bridge condition. A bridge condition is a classification number or a letter describing the bridge state based on which the bridge stock can be prioritised for maintenance and/or repair.

The main problem with current bridge inspection methods is the lack of focus on scour, i.e. the removal of the river bed around the bridge structure due to flowing water. Only a few countries, for example, the U.S., prescribe scour assessment as being mandatory for bridges over waterways. Still, there is no fully standardised method for bridge scour inspection. With scour being the main cause of bridge collapses worldwide, the focus of this thesis was placed on research and development of new scour inspection method(s).

This thesis applied a methodology to analyse existing bridge inspection methods. The analysis proved that most bridge inspections have inadequate focus on scour or they require standardisation and improvement of their rating systems. As a result, new inspection method(s) for Level 1 - designed for simple, single span bridges; and for Level 2 - designed for complex, multi-span bridges were developed.

Both methods were verified on 100 bridges in Ireland. The verification process was based on correlation analysis, detailed pair-wise comparison with other methods and any discrepancies in the results were examined in case-by-case analysis. Verification confirmed that both of the methods (L1 and L2) are applicable on large numbers of bridges. Future training that was set-up during writing of this thesis is an important part of the dissemination and utilisation of the proposed methods to other systems.

Further enhancement of the inspection methods was carried out by integration with a Flood Forecasting System (FFS). The idea behind incorporation of FFS in BMS was to adapt the bridge stock to changing climate and to save resources by operating and planning bridge inspections in a more efficient way. With FFS in place, scour inspections can be scheduled during or after a flood event and not just based on the time of the previous inspection, which required assuming a time interval during which the bridge would remain safe if a flood event occurred.

When it comes to the price of FFS, a budgeting prediction tool was developed as part of this thesis. Based on 11 questions the tool will predict the overall cost of setup and maintenance of a FFS for desired number of years. The tool "PREDICT" is informative and to be taken as an initial guidance for costing of the project.

For bridge inspections, it is estimated that the overall price per bridge inspection is reduced by 81% for L1 bridge scour inspection and between 10-30% for L2 bridge scour inspection. Reducing the reporting time is one of the main reasons for this. By introducing tablet computers for bridge inspections, the reporting time is near zero, enabling bridge inspector to focus only on bridge inspections and reduce time spent in the office.

This work provides bridge scour inspection methods that are verified for practical use. The role of FFS is successfully demonstrated and recommended for use as a standard for BMS resilient to extreme weather events.

List of Publications during PhD

Chapters in a book:

1. *Kerin, I., Sanjay, G., Bekic, D. (2018) "Simulation of Levee Breach Using Delft Models: A Case Study of the Drava River Flood Event" In: Advances in Hydroinformatics, Springer Water, Springer Nature Singapore Pte Ltd., DOI: 10.1007/978-981-10-7218-5_78.*

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Papers:

1. *Bekic, D., Kerin, I., Verkade, J., Gilja, G., Pakrashi, V., McKeogh, E. (2018) "Flood Early Warning System for Supporting Decision Processes on Roads and Bridges" in: Sustainable Bridge Management, Cost Action TU1406 WG3, WG4 and WG5 Workshop, Wroclaw, 1-2 March 2018.*
2. *Cahill, P., Michalis, P., Pakrashi, V., Bekic, D., Solman, H., Kerin, I. & McKeogh, E. (2018) "Development of Bridge Management System and Associated Inspection and Maintenance Procedures for Intelligent Asset Management." In: Proceedings of 2nd International Biennial Conference on Bridge Management (IBMS), Oct. 22nd – 26th, Hyderbad, India.*
3. *Kerin, I., Giri, S. & Bekic, D. (2017) "Simulation of Levee Breaches on River Drava During November 2012 Flood Event Using Delft-FM Model." In: Proceedings of SimHydro 2017, Jun. 14th – 16th, Nice, France*
4. *Bekic, D., Kerin, I., Michalis, P., McKeogh, E. Cahill, P. & Pakrashi, V. (2017). "BRIDGE SMS: Intelligent bridge maintenance and management system." In: Proceedings of "The Value of Structural Health Monitoring for the Reliable Bridge Management", COST TU 1402; COST TU 1406 and IABSE WC1 Workshop, Mar. 2nd – 3rd, Zagreb, Croatia.*
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6. *Pakrashi, V., McKeogh, E., Kerin, I., McAuliffe, S., & Bekić, D. (2016) "Development of the bridge management system under the project Bridge SMS." In: Proceedings of the COST TU 1406 Workshop, Mar. 30th – Apr. 1st, Belgrade, Serbia.*

Presentations & Abstract in conference proceedings

1. *Kerin, I., Bekic, D., McKeogh, E. (2018). "Bridge management and climate change", id. 18901. At: 28th Irish Environmental Researchers Colloquium - ENVIRON 2018, March 26th -28th 2018, Cork, Ireland.*

2. *Gilja, G., <u>Kerin, I.</u>, Michalis, P., Bekic, D., Pakrashi, V. & McKeogh, E. (2017). "Analysis of uncertainties in bridge scour estimation." In: Proceedings of the 15th International Symposium on Water Management and Hydraulic Engineering, Sept. 6th - 8th, Primosten, Croatia.*
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6. *McKeogh, E. & <u>Kerin, I.</u> (2016) "GNSS Meteorology – Ireland National Report." In: Proceedings of 3rd ES1206 Workshop GNSS4SWEC Advanced Global Navigation Satellite Systems tropospheric products for monitoring severe weather events and climate, Mar. 8th – 11th, Reykjavik, Iceland.*
7. *McKeogh, E. & <u>Kerin, I.</u> (2016) "Bridge SMS" In: Proceedings of 3rd ES1206 Workshop GNSS4SWEC Advanced Global Navigation Satellite Systems tropospheric products for monitoring severe weather events and climate, Mar. 8th – 11th, Reykjavik, Iceland.*

<u>Posters:</u>

1. *"BRIDGE SMS – Intelligent Bridge Assessment Maintenance and Management System." At: 15th International Symposium on Water Management and Hydraulic Engineering, Sept. 6th - 8th 2017, Primosten, Croatia.*
2. *"BRIDGE SMS – Intelligent Bridge Assessment Maintenance and Management System." At: EuroScience Open Forum Marie Skłodowska-Curie actions satellite event, Jul. 28th – 29th 2016, Manchester, UK.*
3. *"Intelligent Solution for Inspection and Management of Bridge Scour Risk." At: 14th International Exhibition of Inventions ARCA, Oct. 20th – 22nd 2016, Zagreb, Croatia.*
4. *"WILD device (Weather Information Logging Device) – Innovative solution for weather monitoring" At: 14th International Exhibition of Inventions ARCA, Oct. 20th – 22nd 2016, Zagreb, Croatia.*
5. *"BRIDGE SMS – Intelligent Bridge Assessment Maintenance and Management System." At: Our Ocean to Wealth Conference, Sea Fest, MaREI Opening of Beaufort building, Jul. 10th – 11th 2015, Cork, Ireland.*
6. *"BRIDGE SMS – Intelligent Bridge Assessment Maintenance and Management System." At: 3rd ES1206 Workshop GNSS4SWEC Advanced Global Navigation Satellite Systems tropospheric products for monitoring severe weather events and climate, Mar. 8th – 11th 2016, Reykjavik, Iceland.*

Definitions

Bridge Scour	Removal of river bed material around bridge substructure (piers, abutments, foundations) due to effects of flowing water
Canal	man-made watercourse.
Catchment / Basin / Watershed	an extent or an area of land where surface water converges to a single point at a lower elevation where water join another waterbody like a river, lake, sea, ocean, etc.; surface water can be from rain, melting snow, or ice; usually exits the basin
Delta	land form that forms at the mouth of a river; this is where the river flows into an ocean, sea, lake, etc.; they form from deposition of sediment carried by a river
Floodplain	area of land next to a stream or a river; goes from banks to the walls of the valley; floods heavily
Levee	a ridge of sediment deposited naturally alongside a river by overflowing water; a landing place; an embankment built to prevent the overflow of a river
Meander	bend in a river; forms when water erodes the river banks and forms a wider valley; middle of river does not have as much energy so it deposits silt
Oxbow Lake	u-shaped; formed by wide meander is cut off; creates curved, free-standing body of water
Precipitation	is the liquid and frozen water, including rain and snow, that falls to the Earth's surface.
Reliability	The probability that the required function of the system can be carried out under the given conditions for a given time interval.
River channel	Natural river channel / watercourse.
Scour	a removal of a river bed around bridge foundations and (sub)structure due to effects of flowing water
Sediment	naturally occurring; product of weathering and erosion of rocks; moved by wind, water, ice, or gravity
Tributary	a river or stream; flows into larger rivers or lakes; EX: the Sava river is a tributary of the Danube River

Abbreviations

AEP	Annual Exceedance Probability
ALADIN	Aire Limitée Adaptation Dynamique Développement International
AROME	Applications of Research to Operations at Mesoscale
BIRD	Bridge Information Recording Device
BIRM	US national Bridge Inspector's Reference Manual
BM	Bridge Management
BMS	Bridge Management System is a means for managing bridges throughout design, construction, operation and maintenance of the bridges.
BRIDGE SMS	"Intelligent Bridge Assessment Maintenance and Management System (BRIDGE SMS)" (Grant no: 612517) is a European Commission, Marie Curie 7th Framework Programme funded Project, under the Industry-Academia Partnerships and Pathways (IAPP) call: FP7-People-2013-IAPP. See http://www.bridgesms.eu/.
CIRIA	Construction Industry Research and Information Association, a neutral, independent and not-for-profit body based in UK
CN	Curve Number
COSMO	COnsortium for Small-scale MOdelling
COST TU1406	EU Funded Action "Quality specifications for roadway bridges, standardization at a European level (BridgeSpec)".
CR	Bridge Condition Rating
DANBRO	Danish bridge management system
DOT	Department of Transportation
ECMWF	European Centre for Medium-Range Weather Forecasts
EFAS	European Flood Awareness System
EIRSPAN	Irish bridge management system
EWS	Early Warning System
FEWS	Flood Early Warning System
FFS	Flood Forecasting System
FHWA	The Federal Highway Administration
GFS	Global Forecasting System
HARMONIE	HIRLAM–ALADIN Research on Mesoscale Operational Numerical Weather Prediction in Euromed
HEC	Hydrologic Engineering Center

HINA	The Croatian news agency HINA is public media outlet and the only national news agency in Croatia
HIRLAM	High Resolution Limited Area Model
IABMAS	International Association for Bridge Maintenance and Safety
IAMS	Infrastructure Asset Management System
ICON	ICOsahedral Nonhydrostatic Model
Method A	Colorado Method - USDA Forest Service Scour Evaluation
Method B1	Bekić-Mckeogh scour assessment - Level 1 Qualitative Scour Assessment
Method B2a	UK Highway Agency BD 97/12 - Level 2 Scour Assessment which calculates relative scour depth
Method B2b	NCHRP Qualitative Scour Risk Assessment method using Qualitative Risk Matrix
Method C	Handbook 47 (BSIS or EX2502) scour assessment procedure
Method L1	New Level 1 General Bridge Scur Inspection proposed in this thesis
Method L2	New Level 2 Detailed Bridge Scur Inspection proposed in this thesis
NCEP	National Centers for Environmental Prediction
NDE	Non-Destructive Evaluation
NDT	Non-Destructive Testing
NMS	National Meteorological Services
NWP	Numerical Weather Prediction
PCA	Principal Component Analysis
PR	Priority Rating from Modified BA 74/06
RAIU	Railway Accident Investigation Unit
Sc.CR	Bridge Scour Condition Rating
SCS	Soil Conservation Service
SGOA	Portuguese bridge management system called: Sistema de Gestão de Conservação de Obras de Arte (SGOA)
SHM	Structural Health Monitoring
SMA	Soil Moisture Accounting
SMS	Scour Management System
TII	Transport Infrastructure Ireland
USACE	U.S. Army Corps of Engineers
USDOT	U.S. Department of Transportation
VRS	Vulnerability Ranking Score from Colorado method
WILD	Weather Information Logging Device

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Chapter 1 Introduction

1.1 Problem identification

1.1.1 Bridge management

All major engineering infrastructures throughout the world require continuous monitoring and maintenance to ensure that they remain 'fit for purpose' and comply with health and safety standards. This complex process requires a range of decisions, some critical, in relation to the identification of performance monitoring parameters, sensor equipment, maintenance investment, and Plan of Action. The managers of these assets are increasingly using computer based infrastructure management systems to support their decision making processes [1]. Within a sub-set of these assets, roads authorities around the world face the main challenges of road and bridge management.

After Malahide Viaduct collapse in 2009, Iarnród Éireann (Irish Rail) realised that the bridge inspection methods in place are inadequate [2]. As a fast response to the Malahide disaster, bridge inspections of railway bridges in Ireland to assess scour risk were initiated. The author of this thesis was part of the team conducting this inspections. The experience and knowledge gathered from Irish Rail scour risk assessment programme was the main motivator to remain in the field of the bridge inspections, design and scour risk assessment. The complexity of the bridge management system due to the amount of data required and interdisciplinarity lead the author into research on Bridge Management System (BMS) and how existing BMS can be improved.

A Bridge Management System (BMS) is a means for managing bridge infrastructure during its lifetime (design, construction, operation and maintenance of the bridges). An effective BMS ensures the maximum lifetime of a bridge whilst using the least amount of resources possible. The fact that funds for managing bridges often tend to be significantly lower than for example annual allocation for road resurfacing, the requirement for an effective bridge management system becomes even more important. This requires building inventories and inspection databases, planning for maintenance, repair and rehabilitation interventions in a systematic way, optimizing the allocation of financial resources, and increasing the safety of bridge users. The major tasks in bridge management are:

- collection of inventory data;
- inspection;
- assessment of condition and strength;
- maintenance and repair;
- strengthening or replacement of components;
- and prioritizing the allocation of funds.

The continual bridge inspection - a fundamental of BMS - and the assessment and maintenance of bridges requires a multidisciplinary approach. Bridge inspection system operators must have a knowledge and appreciation of structural engineering, geotechnics, hydraulics, hydrology, materials and transport management.

Due to the vast array of information associated with bridge inspections which must be gathered and analysed to reduce failures and effectively prioritise spending on maintenance on bridge structures, it is no surprise that considerable resources are being invested in finding solutions which strive to simplify and streamline inspection, maintenance and management of these vulnerable bridge assets.

Currently, comprehensive studies and decision processes are primarily the responsibility of a single or a few bridge experts and structures' management personnel. The process is slow and often incomplete due to omissions of key components of bridge integrity, especially without focus on scour [1]. Facilitating, expediting and lowering the cost of management, decision process and planning process can be done by documenting the structure history (status, problems, maintenance and construction works), structure inspection history, monitoring, maintenance, studies and Plan of Action (PoA). This could be achieved by developing a new method and approach which will use innovative ICT technologies, computer models and monitoring equipment. To identify where the focus of a new potentially new method should be, the following section will identify the most common cause(s) of bridge collapse.

1.1.2 Bridge scour as a main cause of bridge collapse

Murrillo [3] indicated that bridge scour is the problem that has caused more bridges to fail than all of the other factors combined. Bridge scour is the removal of the river bed material around bridge foundations and sub-structure due to effects of flowing water [4, 5]. The scour mechanisms with real life examples will be detailed in in Chapter 2 of this thesis. In this section the focus will be on showing the numbers and statistics that support the thesis that scour is the leading cause of bridge collapse worldwide. Damage caused by scour is an important component that affects maintenance of the bridges. Dromey et al. [6], based on 1367 bridge inspections in Co. Cork in Ireland, showed that the scour is the second most frequent damage type recorded on bridge piers and abutments. Data [6] show that the first damage type is loss of pointing at masonry piers and abutments. While damage due to scour could have a significant impact to the immediate safety of the bridge, loss of pointing would not affect bridge safety in the short term. Still, it is an important component in the maintenance of bridges. In the section below statistics on bridge collapses will be given.

1.1.2.1 Statistics on bridge collapses

Research shows that scour is the leading cause of bridge failure in the United States, with 20,904 bridges listed as scour critical nationwide (Gee 2008 [7]). In fact, various research in the last couple of decades confirms that 50-60% of all bridge failures in the US are caused by flood and scour. In his study, Murrillo [3] noted that in a period between 1961 to 1976, 56% (48 of 86) major bridge failures in the United States were the result of scour in the vicinity of the bridge piers. In 1991, Shirhole and Hole [8] report that of 823 bridge failures since 1950, 60% of bridge failures were flood and scour related. In 2003, Wardhana, and Hadipriono [9] showed that 52.88% (266) cases of all bridge failures in the United States (503) from 1989 to 2000 were due to hydraulic factors (this includes flood[1] 32.80% (165), scour 15.51% (78), debris 3.18% (16), drift 0.40% (2) and others 0.99% (5)). As very often scour can be classified as flood factor, flood and scour can be merged into a single class. A continuation of this study was conducted by Taricska in 2014 [10], his analysis showed that 341 bridge failures occurred between the years 2000

[1] Note that very often flood and scour are closely related and it is difficult to distinguish if the bridge collapsed due to flood or scour effects.

and 2012 in the United States, and that scour and flood effects combined were higher than any other single cause of failure, and together, accounted for nearly 50% (153) of all bridge failures over the timeframe studied. Combining the results from the two studies (Wardhana, Hadipriono 2003; Taricska 2014) [9, 10], in the period from 1989-2012 (23 years), a total of 844 bridge failures in the United States were recorded, of which 49.64% (419) failures were caused by hydraulic forces (flooding and scour). The study by Lee [11] confirmed this as in the period between 1980-2012, 501 (53.47%) of total 937 bridge collapses occurred due to flood/scour. The distribution of the cause of bridge collapses in the US is shown in Figure 1.1.

Proske [12] gave the most recent overview research on bridge collapses world-wide, see Figure 1.2. Lee [11], Harik [13] and Cook [14] all confirm that flood/scour is leading cause of bridge collapses. Most of the research from Figure 1.2 is focused on the United States [9-11, 13-16]. The world-wide context of bridge collapses is analysed by Scheer [17], Imhof [18] and Biezma and Schanack. Scheer [17] analysed 107 bridge collapses worldwide and noted five (4.6%) bridge collapses closely related to scour. Imhof [18] analysed 347 bridge collapses and found that most of the collapses occur during construction. This is in contrast to statistics from US, where Lee [11] indicated that 97% of bridge collapses occur during their service life, and only 3% during the construction. Note that the 65% of bridge collapses in study by Lee [11] were steel bridges.

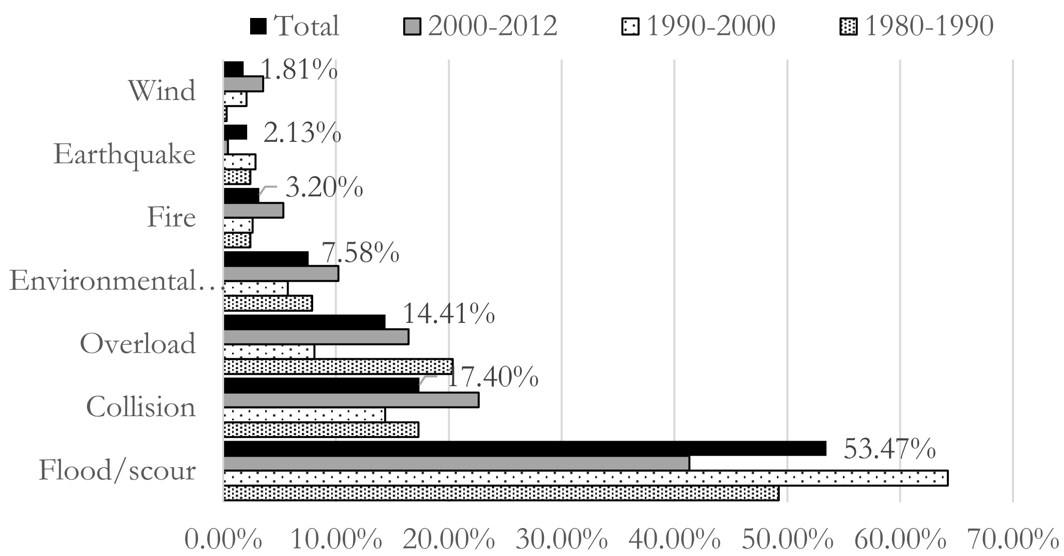


Figure 1.1 Causes of Bridge collapses in the US from 1980-2012 [11].

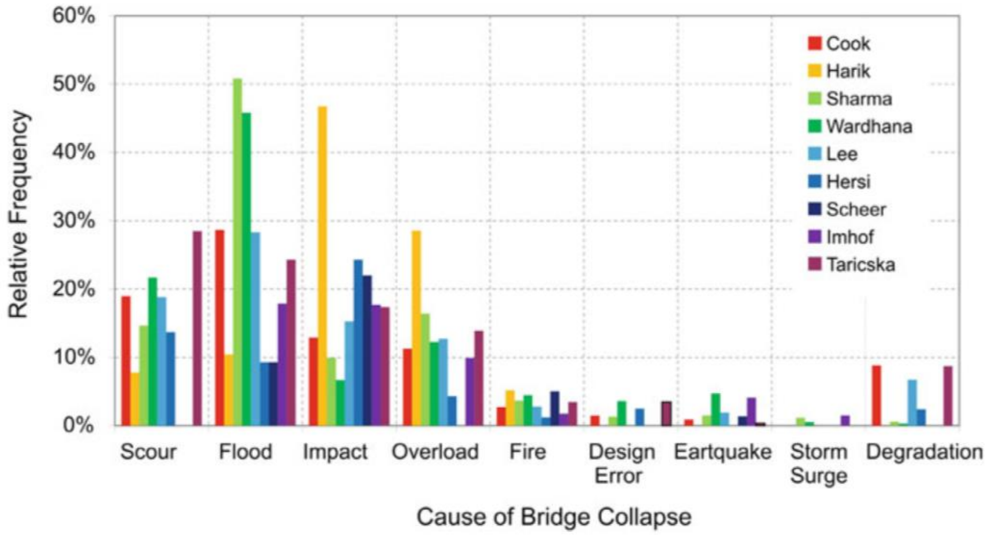


Figure 1.2 Overview of research on bridge collapses according to Proske, 2018 [12].

Biezma and Schanack [19] indicated that that the majority of collapses of bridges spanning rivers occur due to scour. From their research, disasters (earthquakes, floods, avalanches, hurricanes, terrorist attacks, etc.) account 65% of all bridge collapses and scour is the cause of 25% of bridge collapses. It should be noted that there is a close relation between scour and flood events, which means that the percentage of bridge failures due to scour can be higher than indicated. The study [19] is based on a bibliographic research of 350 randomly selected bridge collapses and cannot be considered representative of the overall picture.

According to Melville and Coleman 2000 [20], in New Zealand on average at least one bridge collapse occurs every year due to scour. An overview of railway bridge failures during floods in the UK and Ireland is given in [21-23]. In the period between 1846 and 2003, flood related bridge failures account 131 bridges, 90 of which are can be directly scour related [23]. Furthermore, in the period between 2003 and 2013 an additional 17 scour-related bridge failures [21]. The study [21] records seven bridge collapses in Cumbria, (see Annex A) and the Malahide viaduct partial collapse in Ireland (section 2.3.2), both occurred in 2009. An analysis of 1,400 bridge inspections in Ireland (Co. Cork) showed that bridge scour is the second most frequent cause of damage of bridge piers and abutments, just after loss of pointing [6].

The bridge type and the construction material can be an indication of which hazards a bridge would be more vulnerable to. Bridge stock (type and material of bridges) vary from country to country. Biezma and Schanack [19] indicated that the steel bridges are less prone to scour. The study from Lee contradicts this statement as statistics in the US confirms 65% of bridges collapsed due to scour are constructed from steel. Imam and Chryssanthopoulos [24] showed how the trend of cause of steel bridges collapses is changing, e.g. overall increase in the percentage of bridge collapses due to natural hazards and accidents over the years. They speculate if this could be associated with the effects of climate change. Discussion on the effects of climate change on infrastructure is given in section 1.1.4.

Deng et al. [25] note that beam and masonry arch bridges are the most vulnerable to flood and scour. Taricska [10] showed that concrete bridges have lowest relative frequency of collapse (0.03%) and stone, wood and steel bridge (in order from higher to lower) have significantly higher frequency of collapse (around 0.20%).

1.1.2.2 Condition of a bridge and probability of failure

Some concerning findings about a relationship between bridge condition and probability of failure were noted by Proske [12] in 2018, see Figure 1.3.

Following the idea from Davis-McDaniel [26], analysis for German highway bridges showed that the highest frequency and probability of collapse is for the bridges with fair and satisfactory condition. This could be mainly related to floods (accidental loads), and not to overload and maintenance as noted by Proske [12]. Assuming that the flooding and scour is the most frequent cause of collapse of bridges with “Satisfactory” and “Fair” condition, the one can argue that the condition of the bridge was not appropriately assessed by current inspection procedures. The collapsed bridges would be considered safe and no scour maintenance and repair works occurred. Two questions can be raised. The first is if the existing bridge inspection methodologies account for scour appropriately and second question is if there is a need for a development of new bridge scour inspection procedures.

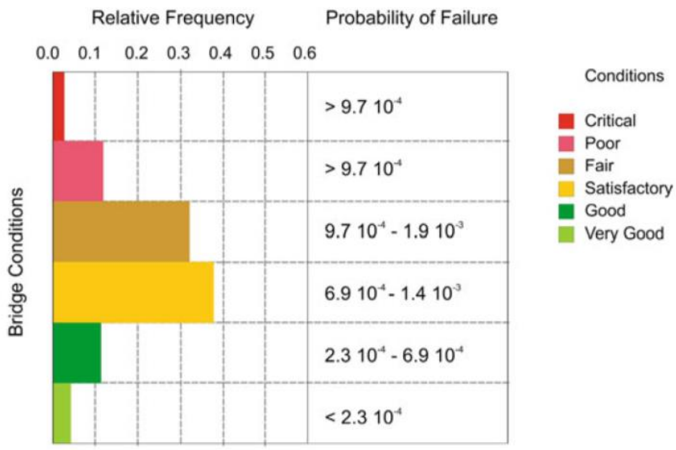


Figure 1.3 Relationship between bridge conditions (for German highway bridges) and probability of failure by Proske [12].

1.1.2.3 Direct and indirect cost of bridge damage

In 1978, the US Federal Highway Administration reported [27] that in the period from 1964 to 1972, damage to bridges and highways from major regional floods was of the order of $100 million per major flood event. Accounting for inflation, this figure would read about $384 million (€326m) per flood event today (https://www.officialdata.org/).

In 2013 Saydam et al. [28] indicated that the maximum total expected indirect loss of bridge collapse (or consequential loss) is much higher than the maximum total expected direct loss. Public data on the collapse of the Malahide Viaduct in Ireland in 2009 support this thesis as the direct cost of repairs was reported to be c. €4.5m - 5m (€10m[2] from Irish Rail source), but the full cost including replacement bus services and loss of revenue was estimated at €10m [29]. In the case of the Malahide bridge collapse the consequential loss was mainly associated with the provision of alternative transport for rail passengers affected by the closure of the Dublin-Belfast line. Consequences of bridge failures were analysed by Imam and Chryssanthopoulos [30].

1.1.3 Bridge Reliability and Ageing of Infrastructure

After the initial high investment cost (construction), the structure has the "as designed quality". This zero state can be confirmed prior to commissioning, by conducting a "zero inspection". The bridge performance, e.g. reliability, would gradually degrade during its

[2] Interviews with Irish Rail Engineers suggests that direct cost of repairs is between €10m-€13m.

lifetime, e.g. due to ageing. Wenzel [31] developed a mathematical formulation of ageing of the bridges that is now implemented in Eurocode: EN 16991:2018. The risk of bridge deterioration cannot be completely eliminated. But a good maintenance programme including regular inspection and proper rehabilitation can slow down this degradation process [19]. In order to maintain the condition and performance of the structure and to ensure a compliance with the standards and regulatory requirements for the structure [32], interventions such as continuous assessment (inspections), maintenance and re-construction (life extension) are required. According to CA TU1406 Report [33], EU Regulations - EUROCODE (2009) provides a definition of reliability level of newly designed bridges, but the same Eurocodes do not define reliability levels for evaluation of existing bridges for the remaining lifetime of the bridge(s). The costs associated with the interventions in response to ageing of the infrastructure can be broken down into fixed and variable cost. A bridge inspection would be a fixed cost. Standards determine the frequency of inspections and the competency requirements of those individuals undertaking the inspections. For a known distribution of asset types, these costs can be estimated fairly accurately [32]. On the other hand, maintenance is associated with the need to correct faults or non-compliances that have been identified by the inspections. The majority of the maintenance costs are stochastic by nature (variable) and cannot be predicted individually. For an asset such as a bridge, where the overall average condition is understood, it should be possible to estimate the frequency of such faults and the associated costs. The maintenance costs of bridges are not yet fully standardised and would often depend on sub-contracting costs which vary considerably.

With time, the extent of, and potential for, performance recovery that can be realised through these interventions becomes less and less as the underlying asset condition degrades irrecoverably. Furthermore, the frequency of such interventions increases as the rate of performance decline increases with time and this means that it becomes more expensive to maintain the asset in a ‘fit for purpose’ state. This is illustrated in Figure 1.4. As the bridge condition deteriorates during its lifetime, age of the bridge and maintenance intervals would be important factors for the safety of the bridge. As a continuation from section 1.1.2.1, statistics on bridge age and frequency of collapses will be shown. Proske [12] compared the findings on frequency of the bridge collapses based on age from different studies [10, 11, 14], see Figure 1.5. Taricska [10] showed that the average age of a failed bridge was approximately 58 years showing that age is an important consideration in overall bridge management. Cook [14] suggests lower average age of a failed bridge of 30 years. Lee suggests that the highest frequency of collapse is to be expected for bridges older than 80 years [11]. By comparing the infrastructure degradation scheme (Figure 1.4) with findings from Cook [14] (red line in Figure 1.5) it can be noted how the rate of bridge failures drops down after c. 30 years of age, c. 55 years of age and c. 75 years of age. This can be associated with three maintenance stages that probably occurred by the age of 45 and 65 years of age or even possibly replacement of the bridge (for bridges older than 75 years). Imam research analyses fatigue and performance of metal bridges [[34, 35]].

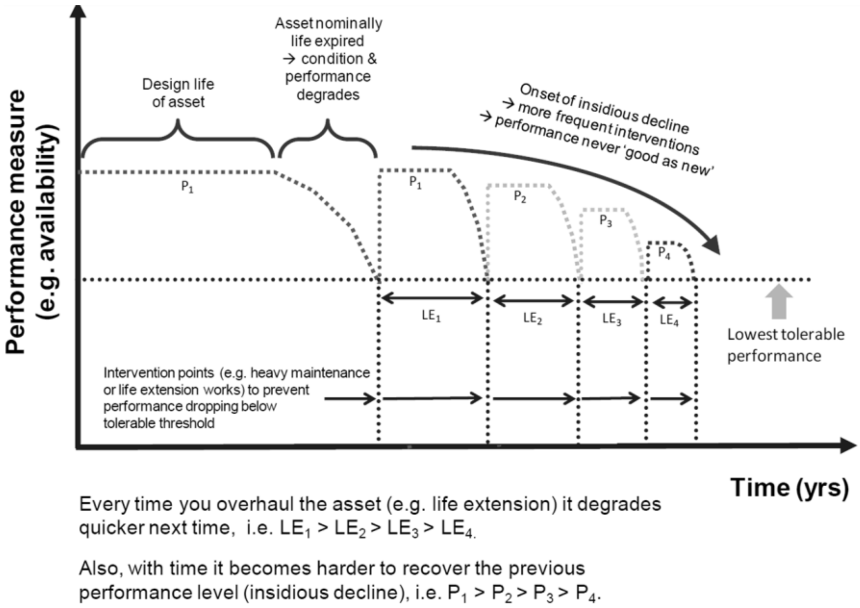


Figure 1.4 Infrastructure degradation scheme [32].

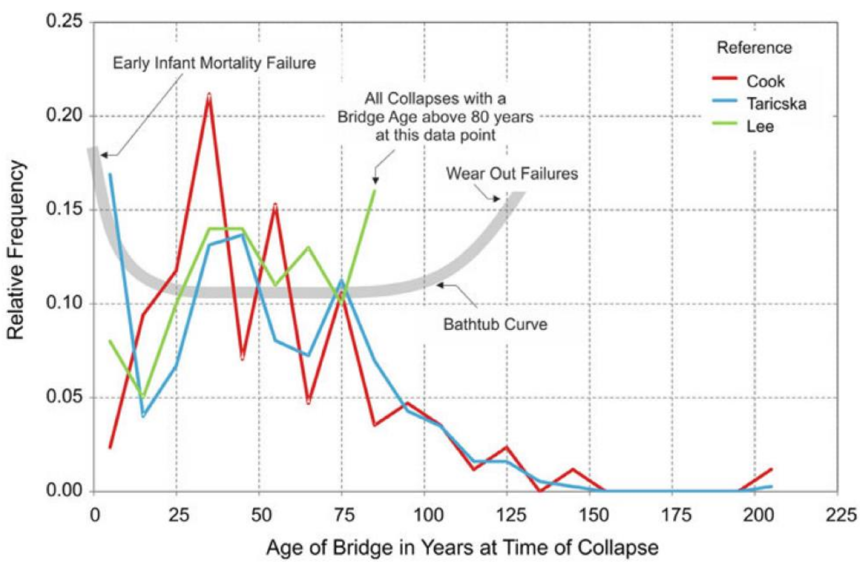


Figure 1.5 Relative frequency of bridge collapse related to the bridge age [12].

1.1.4 Infrastructure Adaption to a Climate change impacts

Global air temperature rising, warming of the oceans, shrinking ice sheets, decreased snow cover, glacial retreat, sea level rise, extreme events, etc. are all indicators of an ongoing climate change [36]. Flood disasters have been increasing over the last 30 years, with over 182 incidents worldwide in 2010 alone [37]. Severe flooding affects more countries than tropical cyclones [38]. Many studies [39-41] report an increase in extreme precipitation. Warmer air has the potential to receive more water content (air humidity) and holds humidity for a longer period of time. Consequently, more frequent droughts and warm weather cause an increase of evaporation [41], forming an endless rising spiral that is difficult to stop. As a result, extreme flood events are becoming more frequent. Fluctuating weather conditions have created a trend of increased rainfall in shorter period of time. Increased urbanisation combined with more frequent and extreme flood events ultimately lead to an increase in flood risk and damage to infrastructure. The EM-DAT dataset [37] shown in Figure 1.6 supports this thesis. According to data [37] from 1900-2016, from all flood related hazards (4721 in record), riverine (fluvial) flooding share is over 54.8% (2588), the flash flood share is 12.6% (597), the coastal flood share is 1.8% (85) and 30.7% (1451) is categorised as other flood related hazards. The rising trend of the number of recorded flood related disasters is shown in Figure 1.6a. According the dataset [37], when compared to other continents, Europe has the highest number (50.8) of flood related disasters per million square kilometres (Figure 1.6b). Interestingly, if we observe the number of disasters per population, Oceania (including Australia) has notably the highest number of 0.42 disasters per million of population. Effects of climate change on bridge scour is studied by Dikanski et al. [42], Imam [43] and Ekuje [44].

The above statistics and the scientific evidence that our climate is changing means that the risk of infrastructural damage will increase in coming years. More resources are being invested in protecting structures from water-related hazards. Future projections indicate that the frequency of extreme flooding across Europe is anticipated to double by 2050 with severe consequences on infrastructure assets [45]. Jongman et. al. [45] suggest that that risk management for increasing losses due to flood is largely feasible. Jongman et. al. [45] demonstrate that the risk can be shared (financing), reduced by investing in flood protection, or absorbed by enhanced solidarity between countries. According The Royal

Academy of Engineers Report [46] "***Government agencies, the public and private sectors and professional engineering sectors across Europe need to come together and proactively meet the challenge of creating a climate resilient infrastructure system***".

Chapter 8, section 8.3 demonstrates how bridge condition and scour inspection intervals can be adapted to changing climate conditions, e.g. to more frequent floods or draughts.

In order to adapt to effects of climate change it is possible to establish a network of monitoring and forecasting systems for bridges. Based on the physics of atmosphere, river geometry and characteristics (land cover, soil) of river runoff area (river catchment), engineers can predict water levels and runoff at the bridges with considerable advance warning prior of the flood events.

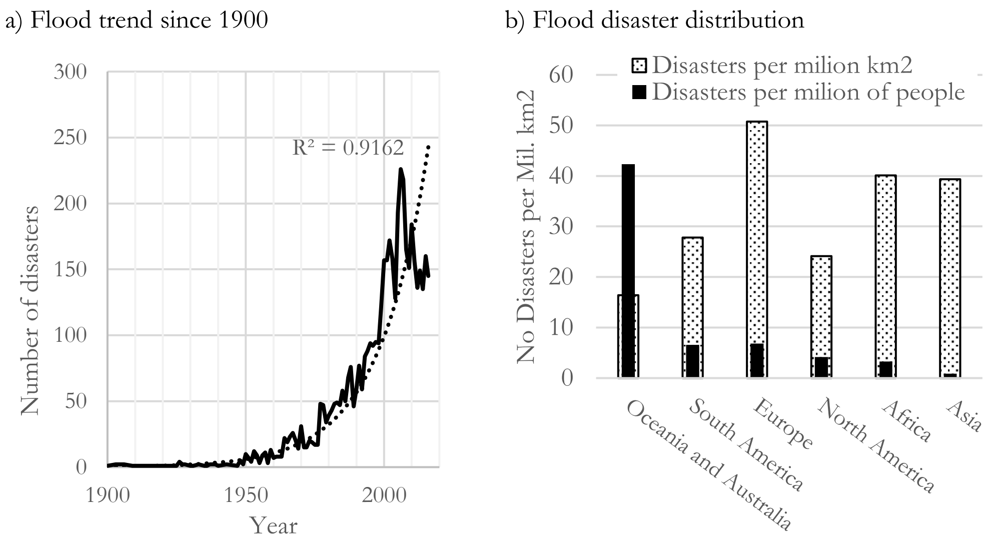


Figure 1.6 EM-DAT: Flood related disasters from 1900 to 2016 [37].

From the perspective of bridge inspection management, for scour susceptible bridges, an appropriate procedure would involve conducting a bridge inspection after a major flood event. In current practice, engineers would rely on a meteorological forecast, not knowing the actual state of water levels at the bridge prior or during the inspection. The combination of a bridge inspection(s) with a Flood Forecasting System would assist the operational management and day-to day, or weekly, planning for the conducting of these bridge inspections. That said, the reader can already see the logic for the implementation and potential benefits of introducing a Flood Early Warning System (FEWS) to the

Bridge Management system. In the context of inevitable Climate Change, Flood Forecasting is becoming a well-recognised solution for flood management as a supporting measure for minimising the risk should preventive or defence measures prove ineffective or are not feasible for implementation. The role of Flood Forecasting in Bridge Management Systems will be discussed in detail in Chapter 8.

This thesis will not focus on ongoing climate change discussion, but will address the problem by placing focus on the need for a 'fit-for-purpose' management system of scheduled or reactive inspections, regular maintenance and timely repair so that the infrastructure elements become more robust to meteorological extremes within a larger, general programme of infrastructural adaption to climate change . To address this problem, a separate chapter - Chapter 8 is presented as part of this thesis.

1.1.5 Corporate memory and human error

The term 'Corporate Memory' was introduced by the Railway Accident Investigation Unit in their report [2] on the Malahide Bridge collapse in 2009 (see section 2.3.2.1). According to the Irish RAIU [2] corporate memory is the knowledge and information from the company's past which can be accessed and used for present and future company activities.

Brady [47] provides three very good examples where elements of human error (wrong engineering judgement, lack of communication, possible personal ego and overconfidence, lack of quality control during the construction due to too high level of trust (between partners) and corporate memory loss (missing of the key information for decision) are analysed. The first two examples refer to failures during the construction of the De Grolsch Veste stadium (7th July 2011, Netherlands) and the Quebec bridge Collapse (29th August 1907, Canada) and would refer to errors in micromanagement, lack of communication and overconfidence. Human factors and errors are indeed important factors in any decision process and will be explained in more detail in section 1.2 and Chapter 9. For now let us focus on the third example, the collapse of Malahide Viaduct (21st August 2009), which shows how the information can be lost over the long history of the bridge. The set of circumstances which led to the collapse of Malahide Viaduct is a perfect example of 'cooperate memory loss' within Iarnród Éireann. The older staff members within Iarnród Éireann had an experience and knowledge on history of changes in maintenance actions and procedures at Malahide Bridge, however after they retired, their knowledge was not transferred and consequently was lost to the organisation. During the later inspections in a thirty-year period leading up to the collapse, the inspectors did not have full information about the bridge condition. In particular, two key facts were missing. These are: the fact that the bridge is founded on a weir (and not in the river bed as assumed) and missing information in the form of the conclusions from the previous inspection report with key findings of the condition of the weir. More details on the Malahide history and the results are shown in section 2.3.2.1.

Furthermore, according to the RAIU report [2], Iarnród Éireann's Infrastructure Asset Management System (IAMS) system had no assigned responsibilities for the uploading of information into the IAMS and there was also no formal process for sharing information when the staff members leave a Division within the company. For example, the reports were stored in hard copy or on the local hard drives with limited access. An appropriate knowledge management system was not in place. The problems were obvious even a few months after collapse during the period of the detailed accident investigation when access to some reports required days of waiting (instead minutes/seconds).

After the collapse, a serious scour problem become more apparent. There was a need for reassessing of the existing infrastructure (bridges) vulnerability to scour. There were two main reasons for this. First reason, as indicated in the report [2], was inadequate training of the staff that was conducting the inspections and the second reason was the fact that there was no standardised bridge scour inspection procedure in place in Ireland.
The Programme included bridge scour inspections for over 100 bridges in a relatively short time.

In a response to the bridge collapse Iarnrod Eireann initiated bridge inspections. Based on initial inspections, some critical bridges were assigned for immediate repairs and maintenance and scour susceptible bridges were recommended for more detailed scour inspection and assessment. The assessment required information on the foundation depth of the bridges. As the majority of the bridges in the bridge inventory had no information on foundations, parallel to the detailed assessment of the scour susceptible bridges, a foundation investigation programme was assigned for a number of bridges. However, the investigation of the foundation depth was not selective and was not coordinated with the detailed bridge scour assessment. Unfortunately, there was no consistency in the bridges that were assigned for detailed scour assessment and for the foundation investigations due to the ad-hoc way in which the inspection decisions were made. The investigation of the foundations was beneficial only for a minority of the bridges that were under detailed scour assessment. This example shows that there was significant room for the improvement of the bridge management in Ireland and showed how different divisions within the same company do not communicate and consequently do not share the same knowledge.

Even today some of the BMS systems are in a transition phase between the "hard copy" approach and modern system that uses all the benefits of the ICT technologies. Following the Malahide Viaduct partial collapse, the lack of a secure repository for information on bridges and knowledge management was recognised. There was an obvious need for a robust and efficient tool that can improve knowledge management within the corporation, lower maintenance and planning costs and consequently to provide more efficient and secure bridge management and operation.

The risk of corporate memory loss, revealed by Malahide collapse (see section 2.3.2.1) and by aftermath scour assessment program of railway bridges in Ireland influenced a decision to initiate an EU research project with a main focus on the Bridge Scour Management System. The funding for the Project was approved in 2015. Effectively Intelligent Bridge Assessment Maintenance and Management System, in future text BRIDGE SMS (www.bridgesms.eu) provides a secure repository for the 'corporate memory' and stirs the pathway for the development of a new bridge scour inspection procedure(s). The author of this thesis started working on bridge scour risk assessment on Irish railway bridges in 2010, after the inspection programme funding ended, author was given a research post and opportunity to work on BRIDGE SMS. The project structure, tasks and deliverables were a major motivation and a starting point for this thesis.

1.1.6 Lack of standardisation

Reports [1, 48, 49] indicate that there is a lack of standardisation in BMS.

Limongelli [49] outlines the main available standards for infrastructure management:

- ISO 55000: gives a framework for asset management;
- ISO 31000: is dealing with risk management framework;
- Eurocodes
  - EN1990 to EN1998: cover the indicators for safety and durability.
  - EN 16991:2018: addressing the risk-based inspection topic. It also contains the mathematical formulation of ageing (degradation)
- ISO 21929-2: focusing on sustainability of civil construction works

Fortunately, the EU Commission recognises the need for further standardisation by funding many bridge management related projects. The COST TU1406 [48] recognises the lack of standardisation for road bridge assessment in the EU. Wenzel and Pakrashi [31] discuss the complexity of the standardisation process and stress that the procedure can take up to 10-years, meaning that the technology could be out of date before it becomes a standard. Limongelli [49] shows examples when it is justified to standardise new assessment methods, e.g. to bring the method to national and international level(s).

The standardisation within this thesis focuses on standardisation of the methodology and not on the implementation of code-of-practice in EU or Governmental laws. That said, the long-term goal of this asset management system development work is the standardisation of proposed methodologies on national and global levels.

In the following section the components of the problem that were identified within the BRIDGE SMS project and summary of the literature review will be discussed.

1.2 Components of the problem

The main components of the problem in BMS, identified following a comprehensive literature review are:

- scour is the leading cause of bridge failure world-wide [3, 5, 7-11, 19-23]
- "The majority of bridge management systems focus mainly on structural issues" [1] without adequate emphasis on bridge scour risk.
- there was a lack of standardisation in BMS [1, 48], e.g. that systems could not be easily adopted by other agencies.
- Lack of resources
- Crisis management Approach (currently)
- Lack of data
- Human error due to a lack of experience or an inadequate brief or as a result of the state of the flow in the river at the time of inspection etc.
- Knowledge based on engineer experience (Knowledge is lost after staff retires) i.e. the loss of 'corporate memory' problem.
- Climate change impacts

The components of the solution are discussed in the following section below.

1.3 Motivation behind this work (components of solution)

Based on the literature review, and the guidance and needs derived from the BRIDGE SMS Project, a thesis that "Introduction of a scour focused bridge inspection procedure would enhance existing structurally-oriented BMS and inspections" is defined.

A second thesis is that "Introduction of Flood Forecasting System (FFS) will adapt BMS to climate change impacts".

The solution will be achieved by developing, standardising and verifying a new method for bridge scour inspections. This way, the existing methods for bridge inspection could be easily enhanced with the hydraulic bridge inspection type for bridges over waterbodies. For the initial implementation, the idea is to keep any of the existing adequate structural bridge inspection procedures and add a separate bridge scour inspection component. Bridges with unknown and shallow foundations would benefit the most of such rapid but detailed approach. A new bridge scour inspection will enable:

- Management of bridges with available resources and subcontracting (split the bridges into categories - simple and complex structures)
- Prioritisation of bridges
- Bridge inspections and monitoring
- Expand existing bridge inspections with more focus on scour issues
- Breaking judgement into multiple components thus spreading and minimising the risk of error.
- Implementation of artificial intelligence (automation) for bridge scour inspections
- Inclusion of flood forecast for more efficient planning of the bridge inspections.

The current bridge inspection methodologies focus mainly on the structural components and condition of the bridge [1]. This often results in remedial works on for example structural cracking without the identification of the cause of the cracks which could be undermining of foundations due to scour. The expected problems, e.g. more frequent floods and of higher intensity, are addressed by the introduction of the new bridge inspection focused on scour (Chapter 6 and Chapter 7). Figuratively, this approach gives us an opportunity to investigate below the water line and see the bridge from this often overlooked perspective.

This thesis will also demonstrate how, by using the modern sensors and prediction models, bridge inspection can be used at its full potential during the post-flood events or even more during crisis management. This will be achieved by a simple DSS model integrating bridge scour inspection results with a Flood Forecasting system (see Chapter 8).

1.4 Structure of PhD and Chapters

After this introductory chapter (Chapter 1) in which the bridge scour is introduced, related bridge scour problems highlighted and thesis and solution to the problems are identified, Chapter 2 explains bridge scour mechanisms in more detail.

In order to address the problems identified above, setting up a framework for the overall Bridge Management System will be discussed in Chapter 3. A cross-view of the Bridge Management System (BMS) requirements with mandatory modules and description of existing BMS is given in this chapter.

Chapter 4 gives more details about very important aspect of any BMS – bridge inspection procedures, with a focus on bridge scour inspection approaches. A description of the most commonly-used bridge scour inspection methods is given. A comparison of identified existing bridge scour inspections is given with pros and cons for each of the method.

Chapter 5 analyses the hydraulic and other river components that would have the most impact on the bridge scour inspection rating methodology. Two methods were analysed as they were applied to a same sample of 100 bridges. A trial for enhancing an existing methodology by developing a new ranking system was done. Although a new rating was suggested for one of the existing methods and by doing so this method was now more transparent, it was concluded that the method still lacked standardisation. The development of a completely new approach would be more feasible in order to standardise a bridge scour inspection by breaking the problem into smaller components and minimising the subjectivity of inspector(s) and reducing the error to a minimum.

Chapter 6 proposes the new approach for bridge scour inspections and describes the method in detail. The method was developed as part of this thesis with the help and financial support of an EU funded FP7 project "Intelligent Bridge Assessment Maintenance and Management System (BRIDGE SMS)" (Grant no: 612517).

Chapter 7 presents an evaluation of a newly developed bridge inspection approach. The evaluation is done based on the application of the new rapid method of bridge scour inspection to 100 previously inspected railway bridges in Ireland. Chapter 7 compares the results of the new inspection with the previous one which was detailed but less standardised in its approach. A comparison of the two methods was obtained by calculaton of R^2 for the results.

Chapter 8 presents a framework on how bridge scour inspection can be used in the more efficient way and how it could serve during crisis management, e.g. flood events. This chapter demonstrates the routes for adaptation of the BMS to a changing climate by introducing Flood Early Warning System into a BMS Decision support system.

Chapter 9 conducts the Cost-Benefit Analysis for the development of the new bridge inspections. Also a simple calculator for estimating the initial and running cost of a Flood Warning Systems as integral part of BMS.

Chapter 10 gives overall conclusions. 0 lists bibliography used during writing of this thesis.

In the following chapter, a comprehensive description of the scour mechanisms will be given.

Chapter 2 Case Studies of Scour mechanisms

2.1 Introduction

Morphodynamic changes of the river channel are a natural phenomenon. By constructing a bridge over a river channel, natural flow patterns and morphodynamic processes are disturbed, restricted and therefore concentrated. The bridge structure usually causes an increase of flow velocities and local turbulence and often undermining of the bridge substructure can occur due to scour. As already indicated in section 1.1.2, bridge scour is the removal of the river bed around bridge foundations and sub-structure due to the effects of flowing water in fluvial and coastal estuarine environments (tides)[3] [4, 5]. The extent of bridge scour would depend on a number of factors, some of which are: the morphologic characteristic of the watercourse (natural river channel, regulated or man-made channel), the material of the river bed and river banks (cohesive or non-cohesive materials such as clay, sand, gravel, etc.), local characteristics of the water section around the bridge (straight section or section in the bend), bridge substructure geometry, skew angle of the bridge, etc.

The above is not a full list, just an indication. The selection of predominant factors (components) that influence bridge scour is analysed in Chapter 5.

Bridge scour that we see and can measure at the bridge is called *total scour*. *Total scour* consists of three types or combination of general scour (vertical and lateral), constriction scour (called also contraction scour) and local scour (being the most obvious to note), see Figure 2.1. The amount of lowering (reduction) of the river bed below the assumed natural bed level is termed *scour depth* [5]. In the following three sub-sections, the three types of scour will be explained with real examples and on a theoretical basis.

[3] Costal environments have additional complexity due to the challenges of wave effects and are classified as coastal erosion. Consequently, there is a distinction between scour and coastal erosion.

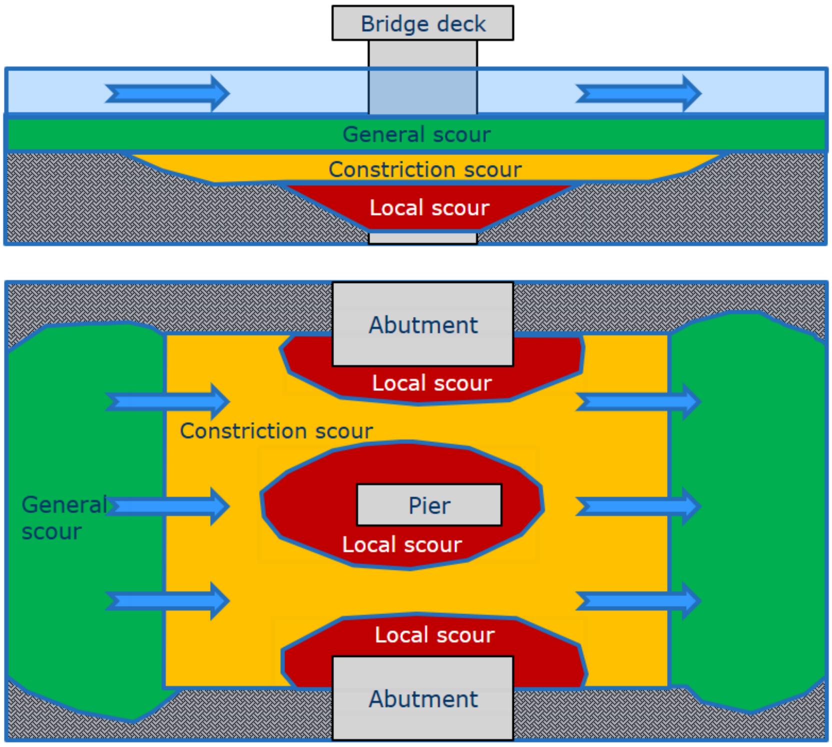


Figure 2.1 Total scour and location of the three types of scour [50].

2.2 General scour

2.2.1 Theoretical background

General scour is associated with the natural morphology of the river, and it is apparent as a lateral movement or a vertical instability of the river. It develops regardless of the presence of the bridge.

Lateral movement of the river is evident in active meandering, an "S-shaped" planform where the channel moves both laterally and downstream [4, 20, 51-53]. A meander is a sharp bend in a sinuous watercourse and is formed when the moving water in a river erodes the outer banks. Meanders are susceptible to continual changes and because of that they are mobile. A classification of meandering channels according to Brice is given in [53]. The flow velocity and water depth would be higher at the outer (concave) bend of the river channel which then leads towards undermining and erosion of the bank, as shown in Figure 2.2. Brown [54] showed mechanisms of bank failures for different bank materials (cohesive, non-cohesive and composite). Although the process of development is similar, local bank erosion should not be mistaken for global lateral migration and meandering. As a guide, the extent of general scour is of the order of kilometres of the river length, whilst local bank erosion is in tens of meters, and does not necessary represent the characteristic of the whole river.

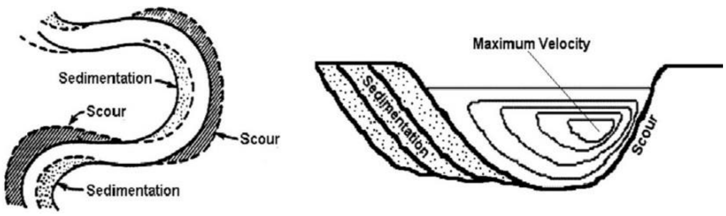


Figure 2.2 Lateral shift of stream caused by bank erosion and deposition [55].

Degradation (lowering of the bed level) and aggradation (rising of the bed level) are long-term changes in streambed elevation due to natural or human-induced causes, which can affect the reach of the river near the bridge [56]. Note that from a bridge scour point of view, aggradation is considered equally undesirable as river bed degradation because it could lead to formation of point bars (inner bend) and cause lateral movement of the river channel.

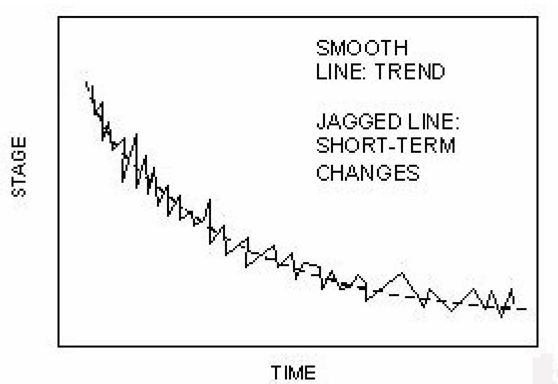


Figure 2.3 Illustration of a specific gauge plot showing stream degradation [56].

Depending on the time taken for scour development, we would distinguish short-term general scour (developed during single or several closely-reoccurring floods) and long term general scour (of order several years or more) [5]. Interestingly, the age of the river can be one of the aspects on how to observe the river morphology. Not only can rivers have different ages, but sections of the same river can have a different age. Miller [57] defined typical sections of a river with different morphodynamic characteristics, see

Figure 2.4. According to Lane [58], young rivers are characterized by their ability to cut their stream beds downward with considerable rapidity and their valley is usually V-shaped with deep gorges or canyons. Waterfalls, rapids or even lakes often exist in young streams. Middle or mature aged rivers are characterised by the U-shape of the river valley. The mature river still cuts the bed downwards but to a much lesser degree than the young river and it also erodes laterally, though not as extensively, when compared to the old age river [59]. The oldest rivers in Europe are the Meuse (320 Mya), Rhine (240Mya) [60] and Thames (58Mya) [61]. Some research [62] supported with and maps [63], suggests that Thames was a tributary of Rhine in Late Middle Pleistocene (Saalian/Wolstonian) era (c0.3Mya). Bentley [64] provided a scheme of a simplified conceptual evolution of river channel and floodplains and Immoor [65] gave an overview of three stages of river development. This concept [65] is not always accepted and sometimes it is considered dated, as it focuses on the erosion only and the deposition and sediment transport processes are disregarded [5]. Therefore, independent of age, it is more appropriate to contemplate if the river is morphologically active or is in equilibrium. Lane [5] defined two pairs of indicators where discharge, Q and longitudinal slope, I, is a first pair and sediment grain size, D_{50}, and sediment transport, Gv, is the second pair of indicators for quantifying the equilibrium or balance of a natural stream, see Figure 2.5. Lane [5] suggests, that if you increase the first pair of indicators Q or I, the second pair of indicators D_{50} and Gv, will need to increase in order to stay in equilibrium and vice versa. If the second pair does not increase in value, the river bottom will start decreasing.

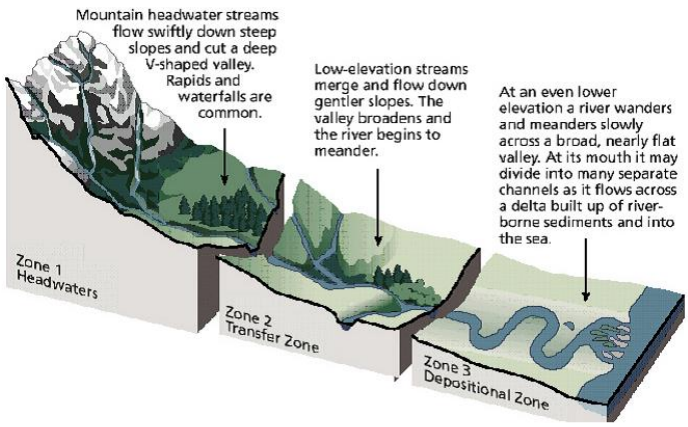
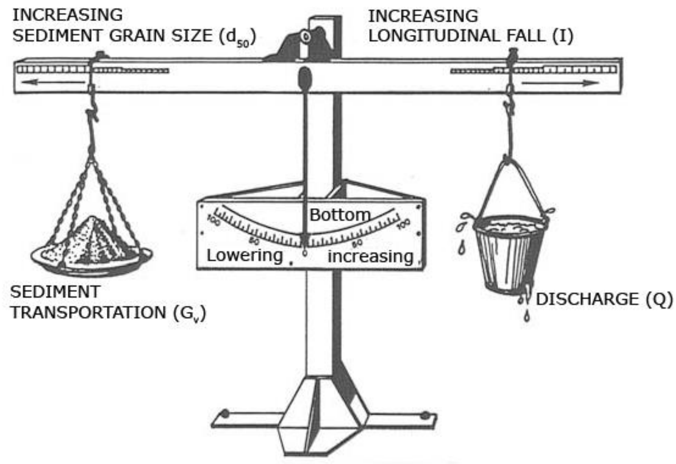


Figure 2.4. River/stream typical zones, source Miller [57].



$$Q * I \propto G_v * d_{50}$$

Figure 2.5 Lane's River Balance [5].

2.2.2 Examples

The vertical and lateral instabilities of the river closely interact and the river can be looked at similarly to Lane's [5] balance concept or even Newton's third law of motion which says that for every action there is an equal and opposite reaction. If, for a natural river channel at its mature or old age (characterised by a lateral movement and no evident degradation), we decide to stop lateral stability, there is a risk that its vertical stability could be disturbed. The opposite effect would occur if we reinforce the river bed at a laterally stable river. This scenario is less probable in practice as usually either only the reinforcement of the river banks or reinforcement of both river banks and river bed would be conducted. In order to better explain this, hypothetical and real case examples will be provided.

2.2.2.1 Railway bridge Jakuševac in Zagreb, River Sava

The railway bridge Jakuševac, constructed in 1968 over River Sava in Zagreb, is an example of a loss of bridge stability caused by global (degradation) and local scour [66]. The loss of stability occurred during an extreme flood event on 30th March 2009 when a freight train was crossing the bridge [67]. According to Gilja et al. [68], the deformation of the bridge structure and tracks occurred as a result of inclination of the south pier due to two main factors: (a) in the period between 1966 and 2009 the river bed lowered around 5-6m (b) the appearance 4-5m deep local scour in area around the bridge pier(s), see

Figure 2.6 below. The superposition of general and local scour gave a total scour depth of 10m at the location of south pier.

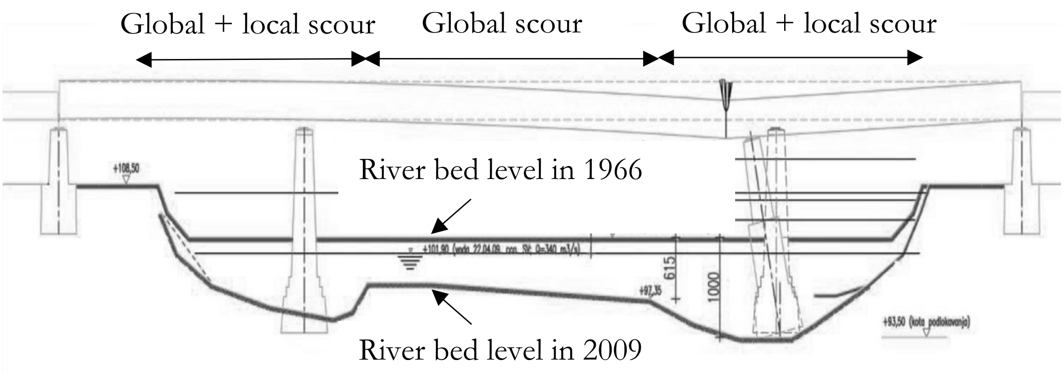


Figure 2.6 Zones of global and local scour and comparison of historic bed levels around railway bridge Jakuševac, source: Gilja et.al. [68] (reprinted with permission).

The degradation of the river bed in the upper reaches of the Sava River (area around the Jakuševac bridge) is a natural phenomenon. However, the degradation process has been greatly increased by human factors: (a) reduction of sediment inflow due to construction of dams and weirs in upstream reaches; (b) shortening of the river course by regulation works (resulting in increase of the longitudinal slopes); (c) gravel excavation from the riverbed. If we consult Lane's balance model (see Figure 2.5) it will be obvious that these factors disturb the balance and result in the lowering of the river bed. The repair works were of order c. 4 million EUR [69].

Figure 2.7 Photograph of near-collapse of Jakuševac bridge (Davor Pongracic - CROPIX).

2.2.2.2 Hatchie River US-51 bridge Failure, Tennessee, USA

The failure of the old bridge US-51 over Hatchie river on 1st April 1989 (Figure 2.8) is an example of how river regulation works (shortening of the channel) can induce instability of the naturally stable river channel.

The old US-51 bridge (west) was constructed in 1936 and the new bridge (East) after 1974. The bridge collapsed prior to the peak of the 2-year return period flood event [70]. The cause of collapse is right bank erosion that undermined the shallow foundations on the floodplain (see Piers 70-72 in Figure 2.9). One can argue that this bank erosion can be associated with a local or even constriction scour, however the channel of the river was shortened and narrowed during the construction of the old bridge in 1936 resulting in natural channel instability (Figure 2.5). Based on the comparison of aerial photographs (from 1948, 1973, 1974, 1976, 1979, and 1984), the Hatchie river channel is reported as laterally stable [70]. However, the same report [70] states that the western Tennessee rivers are unstable and migrate with time and Figure 2.10 suggests old meanders (oxbows) and significant straightening of the Hatchie river around and upstream of the US-51 bridge. This all would lead to a conclusion that a significant man-made alterations of the river Hatchie led to an imbalance of the river and caused vertical and local instabilities of the channel.

To summarise, the failure of the bridge US-51 over Hatchie river on April 1989 is an example of how human intervention (shortening of the channel) caused global instabilities of the river channel, local bank erosion undermining and partial collapse of the old US-51 bridge. The collapse resulted in eight fatalities and significant damage to vehicles (five vehicles fell into the river) and bridge structure (the collapse of three spans and one pier). Further, due to global instabilities of the rivers in West Tennessee, US, bridge inspections are scheduled every two years. The last US-51 bridge inspection occurred in September 1987 and the next inspection was due in six months. Although the channel soundings were obtained and the inspections recommended the design of the scour protection at collapsed pier [70], the cross section at the location of the bank erosion was not plotted nor compared with extent of erosion from previous inspections. This shows how the inspection procedures at the time were not fully adequate and how corporate memory loss (see section 1.1.5) was a factor in bridge assessment and maintenance decision. Similar problems with lateral instability of the channel are noted for State Highway bridge 33 over Homochitto River [71], US-61 bridge over Buffalo River and in Mississippi [27].

Figure 2.8 Collapse of old US-51 bridge (Channel 3 - NEWS 3, Tennessee).

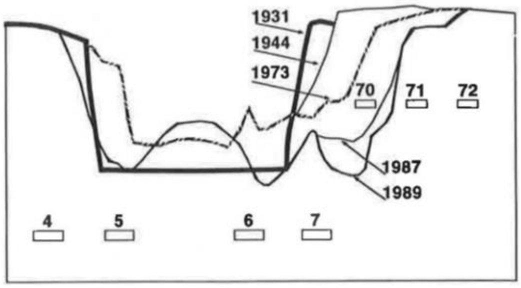


Figure 2.9 Probable channel migration [70].

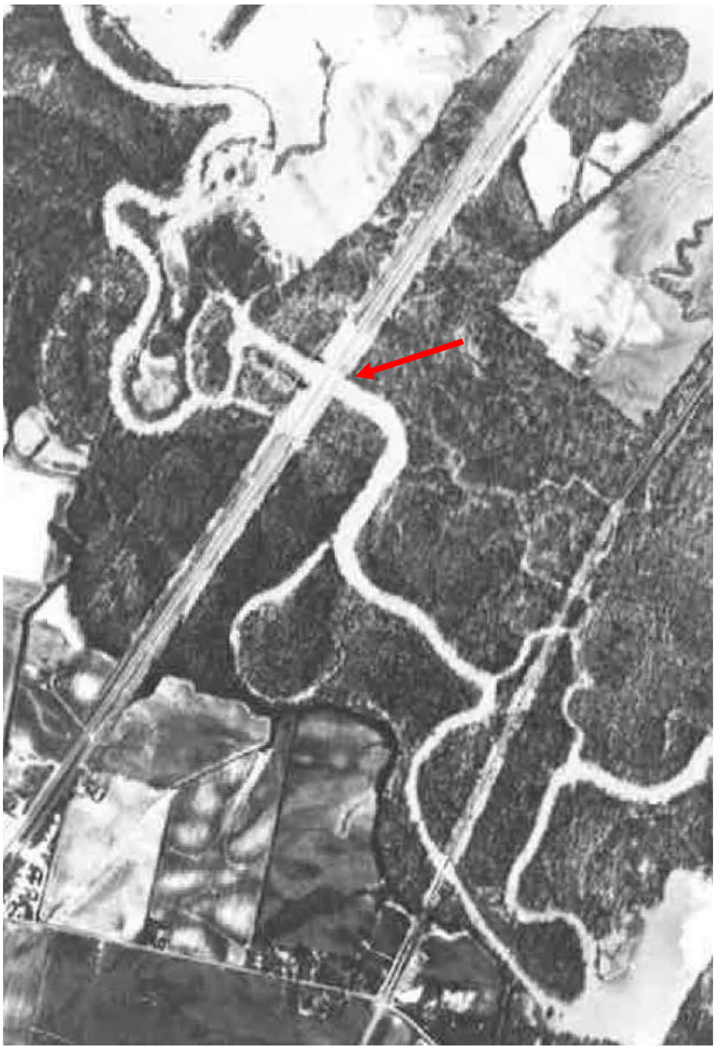


Figure 2.10 Aerial photograph, 6 March 1979 .

2.3 Constriction (contraction) scour

2.3.1 Theoretical background

Constriction scour occurs due to flow contraction caused by the bridge substructure, road/railway embankments or temporary works (causeways and scaffoldings). The bridge substructure acts as an obstacle to the flow [4]. During flooding river bed levels would decrease and after flooding ends some deposition of the river bed is possible (Figure 2.11).

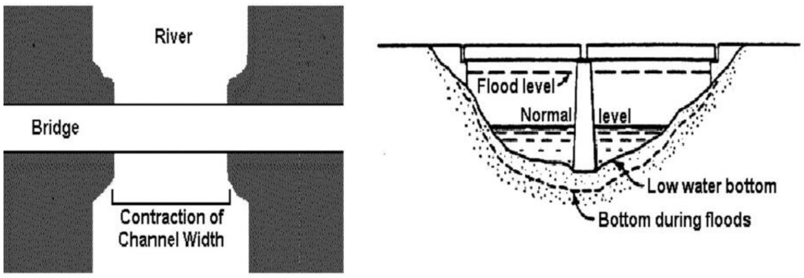


Figure 2.11 Fluctuations of the river bed at bridge profile during floods (degradation) and post flood (deposition) [55].

Constriction scour and local scour (termed as localised scour) very often coincide [20]. Skew angle (angle of the flow attack to pier axis) and debris accumulation could increase constriction even more and also cause additional local scour. The schematics in Figure 2.12 show the layout of the flow trajectories upstream, downstream and at the bridge. By the distance of the trajectories *dy* it is possible to observe the value of flow velocity (the closer trajectories are, relative to each other, the higher the flow velocity).

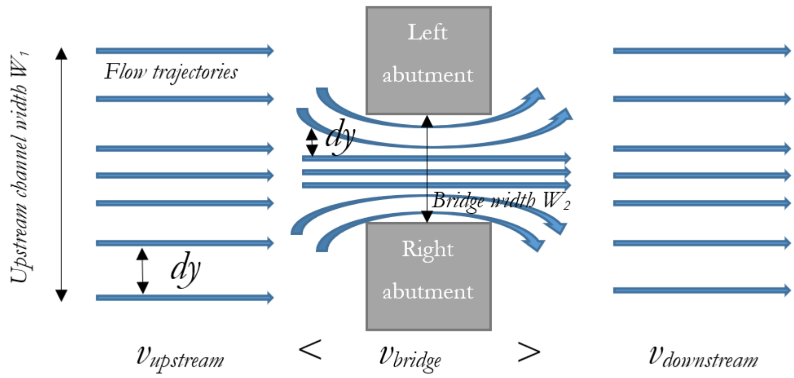
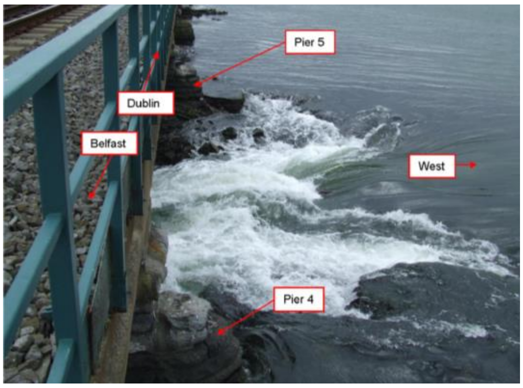


Figure 2.12 Layout of the flow velocity trajectories at the bridge.

2.3.2 Examples

2.3.2.1 Malahide Viaduct, Broadmeadow Estuary, Ireland

Although the Malahide Viaduct collapsed due to local scour, below it will be explained how constriction (contraction) of the railway embankment had a significant role on the flow pattern at the bridge. Also, the importance of the history of the bridge (corporate memory) will be explained. Malahide Viaduct is located on the Broadmeadow Estuary, just north of Dublin, Ireland and it is part of a very busy railway line between Dublin and Belfast. On the 21th August 2009, partial collapse of Malahide Viaduct in Ireland occurred due to scour damage to weir and the undermining of the bridge pier.



a) Photograph taken during inspection of Viaduct on the 18$_{th}$ August 2009

b) Aerial photograph taken after the collapse of Malahide Viaduct

Figure 2.13 Photographs taken closely prior and after Malahide partial collapse, source: RAIU [2].

Only four days before the bridge collapse, the washing-out of some stones around Pier 4 was reported and a bridge structural inspection was carried out only three days before the collapse. Although the weir scour and undermining of the bridge pier (Figure 2.13) was evident during the inspection, reportedly [2] due to lack of training the inspector could not recognise the danger posed to bridge stability. Later the Malahide Viaduct collapse investigation indicated that until that time, the bridge inspection methods used for Irish rail bridges were inappropriate as they did not consider hydrological/hydraulically factors.

2.3.2.1.1 *History*

Since its initial construction in 1844, Malahide Viaduct had problems with scour. A tidal estuary, with a relatively narrow opening for a viaduct, meant that as the tide rose and fell, large volumes of water were flowing through the viaduct waterways. Soon after the opening of the line, scour issues emerged. In the two years following the bridge construction, large volumes of rock were dumped along the line of the structure, gradually forming a rock-filled man-made weir completed in 1846. In 1860 the bridge was replaced with new piers that were since then founded on the weir and not in the river bed. In 1965 the bridge structure was renewed for the third time. In the period between 1967 to 1972 attempts to stabilise the weir with grouting were carried out. The last documented maintenance of the weir was in 1996. An inspection in 1997 reported erosion problems but no significant remedial action was taken.

2.3.2.1.2 *Cause of collapse*

As indicated above, the Malahide Viaduct is constructed and founded on a man-made weir which was prone to erosion. Malahide Viaduct is located on the Broadmeadow Estuary, typically 1800m wide at the location of the bridge. The 1800m of the estuary is contracted to a 176m bridge opening. The Estuary is tidal and due to the narrow bridge opening (10 times less than the natural river channel) high flow velocities from flood and ebb tides are generated, see Figure 2.14. Note that in natural flow conditions (without the bridge and railway embankment) the flow velocities would be significantly lower.

Figure 2.14 Malahide Viaduct construction narrowed the natural channel 10 times.

Due to heavy load and high flow velocities, the weir continued to deteriorate. Weir erosion progressed until the damage of the weir caused the undermining of the pier. Eventually the Viaduct's questionable design (piers founded on the man-made weir), major contraction of the flow due to the inadequate opening of the bridge, inadequate training of staff for bridge inspections, corporate memory loss (the engineers responsible for the structure, including the bridge inspectors were not aware of the 1997 inspection report with a key findings of the weir state and lack of maintenance of the weir resulted in a level of damage of the weir and local scour that eventually caused the collapse one of the of the piers.

The bridge and weir reinstatement soon commenced [72-75]. The full cost including replacement bus services and loss of revenue was estimated at 10 million EUR [29].

2.4 Local scour

Local scour is the most noticeable type of scour and is mainly caused by the obstruction due to the piers and abutments with the flow. Upon impact with the piers, the water particles tend to move downwards, causing the development of horse-shoe vortexes which cause progressive excavation of the material. Scour holes develop immediately at the bridge piers and abutments. For non-cohesive river bed materials, development of the scour hole typically develops upstream of the bridge piers, see Figure 2.15. Local scour mostly is a significant portion of total scour (see Figure 2.1). Therefore, it is often difficult to distinguish total scour from local scour and vice versa.

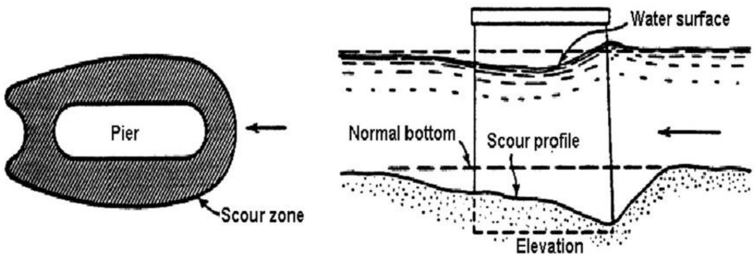


Figure 2.15 Local scour at bridge pier [55].

Theoretical local scour depth D_s [m] can be obtained using one of the empirical formulas for scour depth around bridge piers / abutments: the Laursen's formula (1960) [76]; Neill's formula (1973) [77]; Breusers et al. (1977) formula [78]; the Jain and Fischer (1979) formula [79]; the Froehlich's (1988) formula [80]; the Kothyari et al. formula (1992) [81]; the Melville formula (1997) [82]; the Briaud formula (1999) [83], the CSU formula from HEC-18 (2001) [84]; or other. Depending on the formula used, scour depth D_s [m] is calculated based on the following parameters: Flow velocity v [m/s], shear stress τ [N/m^2], Froude number Fr [1], water depth Y [m], median grain size $D50$ [mm] and bridge geometry [85, 86]. For scour calculations around bridge piers and abutments see sections 8.3.4.2.2 and 8.3.4.2.3 respectively.

The examples of local bridge failures due to local scour are shown in Annex A.

2.5 Conclusions

For most of the cases it is difficult to determine if the damage/collapse of the bridge would occur due to local, constriction or general scour only. The above examples showed that bridge scour occurs in a combination of two or all three types of scour. This is why the use of term "total scour" is more appropriate, see Figure 2.1.

After showing how the scour affects the safety of a bridge (Chapter 1) and after the mechanisms are explained with given examples (Chapter 2), an overall Bridge Management System (BMS) with all modules will be described in more detail in the following chapter (Chapter 3). In this way the reader can understand which modules of the BMS will be enhanced with the work from this thesis.

Chapter 3
Bridge Management Systems

The purpose of a Bridge Management System (BMS) is to ensure traffic safety and maintenance of the bridge infrastructure in the desirable condition during the lifetime of the bridge – from the design to a replacement or demolition, for the lowest possible cost. This is done by conducting an inventory, bridge inspections, monitoring, and allocation of funds for repair and maintenance.

The BMS evolution has been triggered by the challenges posed by ageing bridges around the world. Infrastructure stock or a database on the bridge inventory is the foundation of any BMS. The condition of infrastructure gives a basis for planning of financing and allocation of funds and conducting maintenance and repair works. If the bridge is not in the inventory, or BMS database, there will be no bridge inspections. If the bridge inspection is not conducted, no maintenance or repair works can be initiated, resulting in the deterioration of the infrastructure. In the future, sensors and various SHM monitoring techniques could provide information on the condition of the bridge in the real time. However, BMS still rely heavily on the bridge inspections as the tool for the assessment of the bridge condition. Structures that are not in satisfactory condition are not considered safe for traffic. In order to keep the bridge condition satisfactory and to slow the deterioration of the bridge structure, continuous repair and maintenance works during the bridge lifetime are required. Bridge inspections and sensor readings would trigger an alarm that some works on the bridge are required. Repair and maintenance works depend on available funds, human resources, the amount of traffic, etc.

Before going into more detailed descriptions of the main modules (components) of a BMS, a cross section from a literature review of existing BMS is given in the following section.

3.1 Existing Bridge Management systems

Sorting by countries [87], the most advanced BMS software tools in Europe are found in Sweden, Austria, Denmark, Switzerland and Germany. Screening of bridge evaluation procedures in Europe (Finland, Denmark, Norway, Sweden, France and Germany), conducted in 2008 by US FHWA [88], highlighted an importance of standardisation of bridge inspection frequency, development of guidelines for developing Quality Assessment and Quality Control procedures, illustrations and reference photos for manuals and development of integrated inspection repair approaches. The Everet et al. research on Bridge Evaluation Quality Assurance in Europe [88] contributed to development of the of the US national Bridge Inspector's Reference Manual (BIRM) [89]. A list of recognised BMS databases and software in use in the United States are PONTIS (Preservation, Optimization and Network information System), BRIDGIT (Bridge Information Technology), PENBMS (Pennsylvania Bridge Management System), BridgeWatch® and CAESAR [90], which is focused entirely on the scour risk assessment.

An overview of 25 existing bridge management systems in operation across 18 countries was published by the IABMAS Bridge Management Committee [1] in 2014. The report [1] indicated that all BMS show a strong focus on structural health monitoring of bridge structures, managing this aspect of bridge stability to varying degrees. A list of all BMS noted from the literature review [1, 87, 88, 91, 92] is shown in Annex B. A short description of the most relevant BMS databases and software in Europe is given below.

3.1.1 DANBRO, Denmark

DANBRO [93, 94] is a MS Windows based computer program for all phases of the service life of a structure. It was developed in 1988 for the Danish Road Directorate. The main modules of the DANBRO system may be summarised as follows: inventory module, inspection module, optimisation module for the prioritisation of bridges, budgets and cost control module for the current fiscal year, long term budgeting module and price catalogue module. The outputs of the system are condition reports, including a 5-year budget management based on the Principal Inspections, maintenance plans and economic consequences including direct and indirect costs“ (see section 1.1.2.3 and 1.1.3).

Besides Denmark, DANBRO-based Bridge Management Systems are implemented in various countries such as Colombia, Croatia, Honduras, Malaysia, Mexico and Saudi Arabia (source: [93]).

3.1.2 EIRSPAN, Ireland

EIRSPAN was "developed in 2001 using DANBRO as a starting point" [95]. However, the two systems are not directly compatible due to significant modifications that occurred. The EIRSPAN [96, 97] database is a web-based system used to prioritise maintenance needs of road bridges in Ireland and maximise the use of available funding maintained by Transport Infrastructure Ireland (TII). Routine bridge inspections and maintenance are the basis of the EIRSPAN system. The bridge inspections are divided into routine inspections conducted by the Road Supervisor and Engineer at least every week and year, respectively. An Engineer can issue a request for routine maintenance [96] or a Principal Inspection [97] in case further assessment on bridge condition is required. A Principal Inspection is usually conducted every six years. According to Duffy [95], the maintenance costs are taken from the individual tendered rates for repair works throughout the Ireland.

The ranking of the bridges is between "0" for the best condition and "5" for the worst condition of the bridge. The preliminary ranking of the bridge is calculated automatically based on the bridge general condition and the amount of traffic. The final ranking can be shifted manually depending on other factors, such alternative routes or future plans for the bridge to be by-passed, that are not considered in the preliminary ranking.

The outputs from the EIRSPAN system are the condition rating of the bridge and its components and various personalised reports for individual components or all components of the bridge.

3.1.3 SGOA Sistema de Gestão de Obras de Arte, Portugal

Until 1990's in Portugal, the bridge management was relying mainly on skilled technicians. After the Hintze Ribeiro Bridge collapse in 2001 (see section A.6), the importance of the regular bridge inspections and also underwater bridge scour inspection was recognised which resulted in the development of the Sistema de Gestão de Conservação de Obras de Arte - SGOA system [98], which integrates bridge inventory data, bridge geospatial location and bridge inspections as a main means of identifying required maintenance, repair and rehabilitation works. This means that the bridge inspections are a basis for planning and funding of the required works.

The SGOA system defines six different types of inspections: inventory inspection, routine inspection, periodic inspection, special inspection, extra inspection and underwater inspection. Underwater inspection is further divided into primary, detailed, extra and special underwater inspection. The frequency of the inspections is 4 years for smaller structures and 6 years for larger structures. Further elaboration of underwater scour inspection is done in section 4.1.6.3.

Similar to DANBRO and EIRSPAN, the bridge condition evaluation is made using a component-based system, where 15 components are inspected and rated from zero (no damage) to five (ultimate damage) [98]. Additionally, overall bridge condition rating from 0 to 5 is assigned. The SGOA system recommends that action is needed for condition rating three or higher, while condition ratings lower than three require no action.

3.1.4 HiSMIS, UK

HiSMIS (Highways Structures Management Information System), developed in 1990, became the most widely used BMS in UK [91]. The system is not limited to bridges, but also includes structures such as tunnels, retaining walls, culverts, causeways, flood-ways and fords. The input information of the system consists of five modules: History, Inventory, Inspection, Maintenance/Financial and Programme/study. The system outputs are in form of enquires and reporting contained within six modules: Inspection Management, Works Order Interface, Maintenance Management, Financial control / reporting, Heavy/wide load routing and Replacement and upgrading programming.

3.1.5 BridgeWatch®

USEngineering Solutions Corporation BridgeWatch® is a centralised system which makes all database and geospatial information accessible in a web-based monitoring software. As reported by Young [99], in 2016 BridgeWatch® is used in the United States by seven state DOTs (Iowa, Georgia, Illinois, Pennsylvania, Tennessee, Idaho, and Connecticut). In Europe, BridgeWatch® integrated with Strainstall's Smart Asset Management (SAM)™ system is implemented on the Queensferry Crossing in Scotland [100], see video (www.youtube.com/watch?v=DhfF8Ge8aiw). The same approach, under different branding FloodWatch®, is adopted by the US Geological Survey and National Weather Service in order to provide users with a web interface capable of showing recent and historical river heights, precipitation totals, discharge, and flood stage data from the US Geological Survey and National Weather Service gauge network. For further information see: www.usengineeringsolutions.com. The annual costs of implementation of BridgeWatch® is discussed in [99].

3.1.6 Other BMS in Europe

Helmerich et al. [87] summarised the infrastructure research and Integrated Bridge Management Tools in Europe. The research [87] describes the BMS from Sweden, Austria, Switzerland and Germany:

*“**BaTMan** is the Swedish Management System of the Road Administrations. Other users of this digital system are the Swedish Railway Department and several city and harbor authorities. BaTMan includes all administrative data, technical data of the object; load capacity data and all inspection records. All information is given on repair, strengthening and maintenance incl. their costs.”*

*“**BAUT** is an Austrian Bridge Data Base: Brückendatenbank, BmfwA Wien, Sept. 1999: The bridge data base uses the software BAUT to manage the road bridge infrastructure and to minimize the maintenance costs. The application is set-up upon a modular database system. Various modules have been realized relevant in daily business of running and maintaining a road network. BAUT collects every structural item along the main and secondary road network in Austria.”*

*“**KUBA,** a Swiss Bridge Management System for guidance of all subsequent work was released in 1995. The preservation model in KUBA-MS includes condition assessment and condition forecast, specification of technically plausible actions and elaboration of working program.”*

*“**SIB-Bauwerke** (SIB-structures) was developed and continuously upgraded by the **German** Highway Administration (BASt) on behalf of the Ministry of Transport. The system follows the national standard DIN 1076 and uses links, e.g. for SCMI, Structures Condition Marking Index of UK`s national Network Rail collects data from inspections on forms. The aim is to express the condition of a bridge in a scale between 0 and 100. The SMCI is not a safety index and does not reflect the structural adequacy. The bridge is divided into segments and describes severity (A, B, C..) and extent (1, 2, 3,…) of the worst visible defects. The records of the bridge examiner are imported into a database, where an algorithm processes the data.”*

3.2 Modules of Bridge Management Systems

After an overview of the existing Bridge Management Systems, description of BMS input modules: Bridge Inventory, Inspection Module, Prioritisation, Sensor Network and Prediction module, Planning and Financing; and BMS output modules: Maintenance and repair works and Monitoring, shown in Figure 3.1, will be described.

Note that black arrows in Figure 3.1 show communication process within different modules of BMS.

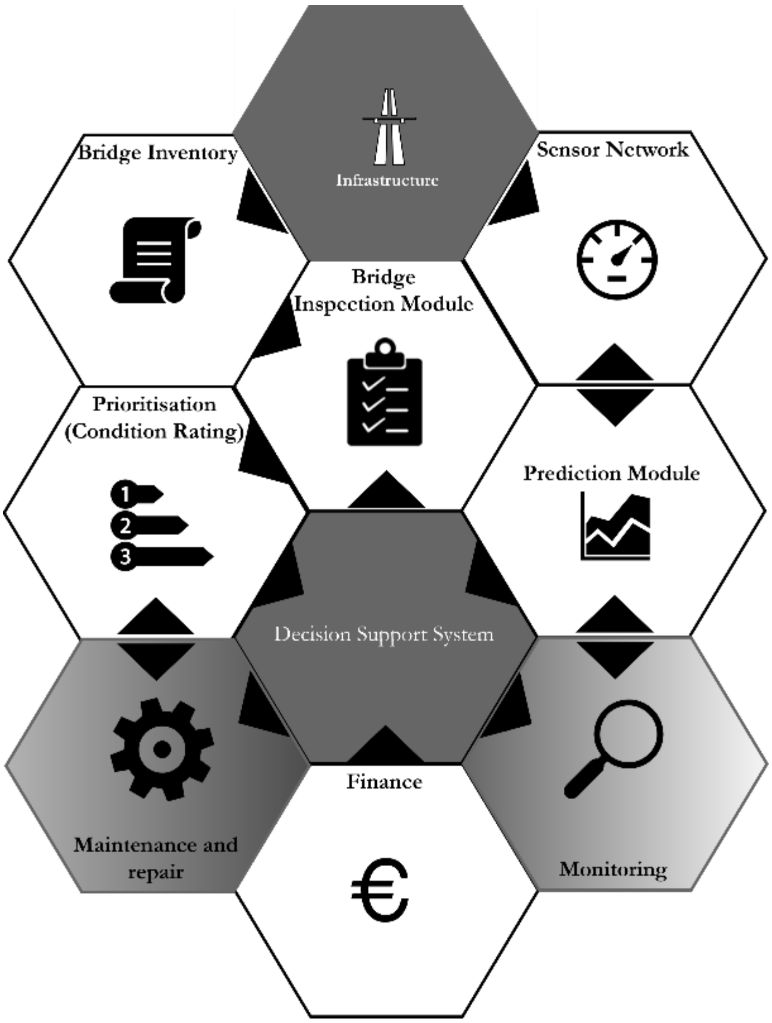


Figure 3.1 Components of Bridge Management System

3.2.1 Bridge Inventory data acquisition and identification of bridges

In order to input the bridge into a BMS, a bridge inventory needs to be collected. Gathering the input data for the bridge inventory is a demanding and costly task, often more expensive than maintenance of bridge management system. The bridge inventory is a static set of information regarding providing general information of the bridge, such as bridge name, bridge ID, geolocation (latitude and longitude), bridge owner, road or railway identification, bridge type, bridge over waterway, etc. For the full list of proposed bridge Inventory, see Annex C. The bridge inventory should assist in classifying the bridges, assigning the roles and responsibilities within the managing agency, and conduct planning on inspections, maintenance and funding.

In creating a bridge inventory, identification, numbering or also referred to as labelling of the bridges is an important and necessary step for unique identification of a bridge within the BMS database and for assisting bridge inspectors in finding a particular structure. Various agencies use different labelling logic. Even solely in Ireland and UK, the labelling differs within the different agencies. The Highway Agency in UK uses kilometres for the numbering of the bridges [101]. Irish railway identifies bridges using under/over structure type[4], line name and number of the bridge on the line. The identification of the bridges for regional and local roads in Ireland uses local authority code, road name, structure and label number, similar to as for road identification [102]. This leads to a conclusion that unification of bridge identification on European level would be of benefit for the development of a standardised bridge management system.

[4] The structure type refers as under bridge (UB) if the obstacle is located under the railway tracks and over bridge (OB), usually for the pedestrian bridges if the bridge is used to carry people over the railway tracks

3.2.2 Bridge inspection module

The purpose of the bridge inspections is to ensure the safety of the bridge by accessing the current state of the bridge, identify any maintenance, repair, and rehabilitation works that need to be conducted and to prioritise bridges based on their condition in order to provide a basis for planning and funding of the required works. Bridge inspection is the most common Non Destructive Evaluation (NDE) technique for bridge condition assessment. Consequently, most BMSs still rely heavily on bridge inspections.

Bridge inspections are usually visual and therefore qualitative by its nature. They are also not necessarily consistent or standardised. This means that the inspectors may naturally overlook certain structural problems, especially in parts of the structure where the access is difficult. The way to overcome the inspector's subjectivity can be achieved:

1. by breaking judgement into more components rather than one (spread risk of error)
2. implementation of artificial intelligence for bridge inspections based on the input data for the bridge components
3. by introducing quantification using non-destructive testing (NDT) methodologies

The main disadvantage of all bridge inspections in use is the lack of emphasis on bridge scour risk. The majority of bridge management systems focus mainly on structural issues [1]. This work will greatly cover the improvement of the current bridge inspection and monitoring procedures with a strong focus on a bridge scour. The overview of existing bridge scour assessment procedures is shown in Chapter 4.

3.2.2.1 Inspection types and frequency

Generally, all BMS consist of following types of inspections: superficial, general, principal and special inspection [91], in increasing order of the inspection frequencies. Superficial inspection, often referred to as routine inspection, is conducted by road patrol technicians on a weekly basis. General inspection is a visual inspection which is conducted for a simpler structures and usually uses trained staff within the management agency for conducting the bridge inspection. Principal Inspections are more detailed inspections used certified engineers, usually conducted every 6 years and often outsourced. Special inspection, including underwater inspection is a specific inspection that can include Non Destructive Testing (NDT) and Structural Health Monitoring (SHM) techniques tailored for a specific bridge. This type of inspection involves various experts from different fields such as structural, geotechnical, hydraulic engineering, materials and transport can involve significant mechanical and chemical testing of the structure. The cost of special inspection can be significantly higher than principal inspections, depending on the requirement of tests and the size of the bridge [103]. In most of BMS, the scour inspection is part of special underwater inspections. For a bridges over water, there is an obvious need for inclusion of bridge scour inspection in general and principal inspections. The US national Bridge Inspector's Reference Manual (BIRM) [89] provides a good description of inspection types. BIRM is used as a base for each DOT in the United States to develop its own bridge inspection manual. An overview of the inspection types, frequencies and training within the existing BMS is shown in Table 3.1. Study [88] reports how most management agencies in Europe believe that inspector qualifications and experience requirements by agencies allow inspectors to determine the duration between cycles of inspections, typically up to 5 or 6 years but up to 9 years in France. The state-of the art approaches calculate required frequency for next inspection automatically, however, many agencies still allow manual correction of condition ratings and time to next inspection.

Table 3.1 Literature review on Bridge Inspection types, frequencies and duration.

Country	Inspection type	Training provided and duration
Denmark (Danbro)	1. **Road network inspection** (1-3 times a week) 2. **Routine maintenance inspection** 3. **Principal inspections** (few months – 6 years) 4. **A special inspection**	Unknown
Finland (Finnra)	1. **Acceptance inspections** after completion of construction (once) 2. **Annual inspections** 3. **General inspection** (5-8 years depending on bridge size and condition) 4. **Basic inspection** (general inspection supplemented with a variety of tests and core samples taken by the Research Centre of Finland (VTT)) – 5 years 5. **Special inspection** 6. **Underwater inspection**, (5-year intervals) 7. **Intensified monitoring**	Yes, bridge inspector training is arranged by the Finnish Road Administration • 3-4 day theoretical course + 1 day on site training • Finnra provides a 2-day course in bridge register use that must be completed before an inspector is granted rights to update the data.
France	**1. Routine visit**, **2. Annual inspection** (annually), Image **3. Evaluation of quality of the works** (de la Qualité des Ouvrages - IQOA) (every 3 years), **4. Detailed inspections** occur every 3 to 9 years, but typically every 6 years	Six modules of training. The first five designed for bridge inspectors and the sixth required for project manager certification. Module 1: A 6-day course on basic knowledge (strength of materials, reinforced concrete bridges, common steel bridges, common pre-stressed concrete bridges, masonry bridges, culverts, common retaining walls) Module 2: A 1-day course on large pre-stressed concrete bridges Module 3: A 3-day course on uncommon retaining walls Module 4: A 2-day course on large steel bridges and cable bridges Module 5: A 3-day course on tunnels and underground structures Module 6 is a 3-day project manager's course including Methodology of detailed inspection, Investigation techniques, Monitoring and surveillance, Repair and strengthening, Actions after an inspection
Germany	**1. Superficial inspections** is visual inspection performed quarterly for all visible components and annually (all accessible components) by maintenance personal without special knowledge of highway structures. **2. Routine safety monitoring** is performed on an ongoing basis by maintenance personnel as part of their routine superficial inspection of the highway **3. Major inspections** involve visual inspection and testing (material investigations) of all parts of a structure by inspection engineers. (every 6 years). **4. Minor inspections**, conducted every 3 years, are visual inspections by inspection engineers to check the results of the major inspection. **5. Ad hoc inspections** are performed by engineers to obtain an in-depth view of a particular damage or deterioration process that has occurred at the bridge (accidents, flooding, etc.). **6. Inspection in accordance with other regulations and** standards may be required of machinery and electrical equipment forming part of highway structures, especially movable facilities and gantries	professional development seminar for bridge inspectors (5 day course) Planned: • Development of a curriculum leading to a designation as engineer of inspection • Periodic re-examination for renewal of a certificate • Development of training programs for technicians
Ireland EIRSPAN	**The Roads Supervisor inspection** (at least once a week) **The Engineer inspection** (at least once every year or after an event of significance - flooding or collision) **Principal Inspection** (issued if Engineer inspection shows serious damage and is uncertain of its consequences) **Special Inspection** (upon request by a private consultant, in order to deter-mine in detail the nature, extent and cause of damage to a structure.	Regional bridge managers are responsible for the training of the Engineers in the EIRSPAN Routine Activities. Principal-inspection training requires a two week course (Duffy [95]).
Norway (BRUTUS)	1. Major inspections (conducted at least every sixth year) 2. General inspections (typically on an annual basis).	in-house and consultant inspectors based on education and experience
Portugal SGOA	**Inventory Inspection** (first inspection of a new or existing structure) **Routine Inspection** (Every 4-6 years) **Periodic Inspection,** **Special Inspection** **Extra Inspection, and** **Underwater Inspections:** *Primary underwater Inspection* (every 5 years) *Detailed Underwater inspection* (upon request or every 10 years) *Extra underwater inspection* (after damage) *Special Underwater Inspection* (in case that Primary inspection requires more data)	Unknown
Sweden (Swedish Road Administration)	**1. Major inspections** conducted at least every sixth year). inspector decides at the site when the next inspection shall be performed. **2. General inspection** is to follow up on damage identified during the last major inspection and repaired or corrected. **3. Special inspection** may be routinely performed for mechanical and electrical equipment on movable bridges. Special inspections are also performed whenever a regular inspection has indicated a need to investigate in more detail a stated or presumed damage. Normally, only the specific damage or deficiency is investigated.	Unknown

3.2.2.2 Visual inspection and other Non-Destructive Evaluation approaches

Most BMS rely mostly on a visual inspection of bridges, as they are considered an adequate and cost-effective Non-Destructive Evaluation (NDE) method for bridge management. Inspector subjectivity in visual inspections poses the main source of error and is the main disadvantage of the visual inspections. Figueiredo et al. [98] gives a critical commentary on a condition rating system used in Portugal. According to them, the rating based on the visual inspections depends highly on human-based evaluation and the ratings do not exhibit a high degree of consistency when performed by different inspectors. As an alternative, use of Non-Destructive Testing (NDT) techniques such as Schmidt/rebound hammer test, Ultrasonic pulse velocity testing, Radiographic testing, Infrared thermography, ground penetrating radar or other methods are recommended (see page 37-38 in report [98]) and also Structural Health Monitoring (see section 3.2.5) is recommended in order to overcome inspector subjectivity by comparing the components based on physical quantification of the states over quantitative comparison.

Quirk et al. [104] assessed value of information from visual bridge inspections for bridges in Portugal and Ireland, Co. Cork. The cost of the bridge visual inspection is estimated at value of €500, and the results suggest significantly higher benefits of the visual inspection, of order up to c. 13 times higher than the cost of the inspection. When the bridge inspector subjectivity, rated as optimistic, neutral or pessimistic, was introduced, the results showed the variation of ±26% when compared to value of bridge inspection conducted by a neutral bridge inspector (€6,876). The value of visual inspection is very low for bridges with lowest (no action required) and highest (immediate action required) condition rating respectively.

Moore [105] conducted an experiment where he observed results of 10 discrete inspection tasks on seven bridges in United States. Overall 49 bridge inspectors from 25 States were involved in the experiment. Experiment showed high percentage of error, between 48%-58% percent, of the individual Condition Ratings for the primary elements. Also a high percentage of inconsistency when comparing the ratings is noted. Only 68% of the population would vary within approximately one rating point from the average.

A similar exercise was conducted within this PhD work with Cork County Council staff. For details, see Annex D.

3.2.2.3 Outputs: Reporting, condition and maintenance list

The main outputs from bridge inspections are reports, which include photographic documentation, comments, component states, condition rating of the bridge and recommendation for maintenance and monitoring.

The bridge condition rating, usually in the form of a number, is a qualitative description of the bridge's overall state. During the inspection, trained experts are attempting to determine the states of the bridge elements and components, based on their subjective opinion. The bridge inspectors do not conduct any calculations in order to assign element and component states, which are, usually, numbers or letters and describe the relative condition of the element from good to bad. The states of the elements, in the form of numbers, with other factors from the bridge inventory are used as a basis for calculation of the bridge condition rating.

After finishing the inspection, an obligatory part of the report, in addition to the condition rating of the bridge, includes the recommended time to next inspection. Time to next inspection can be automatically calculated based on the bridge condition rating. Many Bridge Management Systems have an option to change the recommended condition rating and time to next inspection manually.

3.2.3 Prioritisation of bridges

The prioritisation process is a process in which bridges are sorted into categories which describe the need for action at the bridge from more immediate to less immediate, based on the priority list (usually ranked as Good, Fair and Poor), planning of maintenance and repair works, monitoring, plans of actions (PoA), closure of the bridges, etc. in accordance with available funds can be approached. Note that the prioritisation process does not necessarily mean that only bridges in a poor condition should be repaired. Many BMS would allocate funds for preventive maintenance of the bridges in good or fair conditions, see section 3.2.4.

3.2.3.1 Scoring system

A scoring system gives a qualitative description of the bridge in form of an integer value. Calculation of the integer for the description of the bridge condition requires acceptance of some uncertainties, such as inspectors' subjectivity (see section 3.2.2.2). Uncertainties in bridge condition assessment are analysed by Deshmukh and Bernhardt [106]. A standardised scoring system is the prerequisite for an efficient Bridge Management System. Validation and verification of a scoring system requires application of a scoring system on a larger number of bridges as a representative sample. After the scoring system for components is defined, bridges' ratings are comparable with each other. If the bridges are comparable, they can be prioritised into a descending list from the bridges for which the action is required immediately to bridges where no action is required.

The procedure for the development of a scour system is shown in Chapter 5. The proposed scoring system and component states classification for the newly developed inspection module is described in Chapter 6.

3.2.3.2 Prioritisation process

Larger networks require prioritisation as not all bridges can be repaired at the same time, or within the same fiscal year. The prioritisation is conducted in order to determine which set of bridges should be repaired first. Usually the repairs would start at the bridges which require the most immediate attention (that are under a risk of a collapse) and then attention would be given to the bridges that are under lower risk of collapse.

The prioritisation of the bridges is made based on the component states and condition rating. Bridge inspection gives a condition rating of a bridge (number) based on which initial prioritisation of bridges can be made. Such approach is used in DANBRO and EIRSPAN system (see section 3.1).

Prioritisation list of the bridges is used for a short and long term planning of maintenance and repair works and for allocation of funding.

3.2.4 Maintenance and repair works

Maintenance works are minor works, for repair of minor defects at and around the bridge for which further studies are not required. Repair works are needed when major damage is present. Prevention, or maintenance is more cost effective manner of damage repair than, more extensive, repair works. Timely repair of minor damages can aid in extending the lifespan of both the bridges and prevent progression of deterioration of the bridge structure. To enable timely and cost effective maintenance and management of the bridge network, it is advisable to provide guidance from the inspection as to required typical maintenance and repairs for each bridge. Should maintenance works fail to prevent further deterioration of the structure, repair works or even bridge replacement are required. Maintenance plans for the bridges are designed according to the available funds and urgency to ensure safety traffic over the bridge(s).

Bridge inspections and prioritisation processes do not only focus on the bridges that are in poor condition. They should provide information necessary for allocation of funds for planning of the maintenance works, minor defects, for the bridges good and fair condition as well. In the case that further investigation should not confer additional benefit, an immediate maintenance works could be a more cost-effective measure for the bridge safety than setting-up a monitoring procedure or conducting additional studies.

A list of typical works, including structural and scour measures, is shown in Annex E. The following section will focus on scour routine maintenance works and mitigation measures.

3.2.4.1 Routine Maintenance Works for Scour Prevention

When identified at an early stage and carried out in a timely manner, routine maintenance works may reduce the need for major future repairs. These works take into account debris removal, filling in smaller scour holes etc. Problems which can be resolved by routine maintenance works are listed as follows:

- If the debris is not removed at an early stage it will accumulate, which will constrict the flow and increase the potential for scour damage to the structure, the bed, and the banks.

- Minor degradation of the bank protection systems (e.g. collapse of stones positioned upstream/downstream of the abutments for bank protection) needs to be maintained as it can cause further progression of erosion and slope instabilities in the area adjacent to the bridge.
- In order to ensure accessibility to the bridge, vegetation and trees around the bridge should be periodically removed. Vegetation may also contribute to flood debris and tree growth which can result in damage to the structural elements.
- Sediment deposition can have a major effect on the waterway area and consequently lead to damage of the bridge structure. Periodic monitoring and removal, or channel dredging, of deposited sediment in excess of 30% of the channel width area of the needs to be removed.
- Drainage systems/weep holes that are designed to relieve the hydraulic pressure behind structural elements (i.e. bridge, embankments and/or bank protection systems) need to be cleared as required.

Besides the above listed minor maintenance works which mainly focus on the mitigating of the scour potential, more intrusive measures for scour prevention can be used. These are named as scour protection works or armouring, hence re-conditioning or increase of gradation of the river bed and banks. These works are elaborated in the following section.

3.2.4.2 Scour protection works

For bridges where the structure has deteriorated to such extent that in the foreseeable future, maintenance and repair works would not be feasible, partial or complete replacement of the bridge could be considered in order to ensure the safety of the traffic. For the bridges where the structure of the bridge is in satisfactory condition, but with evident scour issues, the following measures could be applied:

1. *Scour reduction measures* to improve flow conditions at a structure, thus reducing the magnitude and effects of scour, e.g., streamlining of piers, streamlining the channel through the bridge waterway, river training, deflectors such as guide banks, sacrificial piles, etc.
2. *Structural measures* to withstand the predicted depths of scour, which in the case of remedial measures include underpinning foundations, reinforcement and extension of foundations, other options such as 'bagged' concrete, sheet piling, concrete grout, etc.

3. *Scour protection measures* to limit the extent to which scour can occur, using 'flexible' systems such as riprap, rock-filled gabion mattresses, or concrete block revetments and similar 'rigid' systems, or so called 'bio-technical' solutions to stabilise river banks.

In the following section scour protection measures, also called armouring, will be discussed.

3.2.4.3 Selection of the type of armouring

The selection of an appropriate type of armouring is dependent on numerous factors such as the erosion or scour mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs.

There are two main categories of armouring: flexible and rigid systems. *Flexible systems* can cope with some movement, without losing their armouring capability and so can adjust to settlement or movement of the underlying and adjacent surface or bed. Such systems are susceptible to failure from movement of the armour material, either because it is undersized or because of loss of material at its edges. The flexible systems can accommodate larger changes in channel stability than rigid systems, and are preferred where there is significant channel instability. *Rigid systems* cannot adjust to changes in the underlying surface and are often impermeable. While normally more resistant to erosion, they are susceptible to failure by undermining and uplift (seepage pressure). The rigid systems are generally more resistant to surface erosion, so can provide good protection against high velocity and high turbulence. The designation of the CIRIA [4] factors for flexible systems shows the following:

1. *Rip-rap* - It shows the widest applicability and low construction costs. It may not be appropriate if the access/headroom to the site is restricted. It is recommended for high velocity flow.
2. *Gabion mattresses and sacks* - This type is less likely to be recommended if there is a need for underwater construction. The construction and maintenance cost of gabions is moderate. Also, gabion mattresses and sacks are less recommended for high flow velocities.
3. *Gabion boxes* - These are not applicable for underwater construction, and are therefore not recommended.

4. *Articulated concrete blocks* – This method has relatively high construction cost and lower resistance to high flow velocities.
5. *Articulated grout-filled mattresses* – This method is applicable for underwater construction and in restricted access areas, and has a relatively low maintenance cost. It is less appropriate for high flow velocities.
6. *Bituminous systems* are not applicable for the underwater construction.
7. *Biotechnical solutions* are not applicable for high flow velocities.

The cost of the system is dependent on various factors, including availability of materials, such as rock, the length of haulage routes to the site, and the type of access available for construction. In general, the systems incorporating concrete are more expensive, unless, there are long haul routes for rock. The cost of construction underwater tends to be considerably higher than construction on dry land.

Working underwater presents particular problems with regards to quality control and health and safety. Systems using concrete as their main component, such as grout-filled mattresses, often provide an effective solution, particularly for repairs where access is restricted and where the repairs need to fill an irregular shape. Perhaps more important, however, is the effectiveness of the measure selected in performing the required function. In the selection of the type of armouring two methods from the UK and the US are used, both described in Annex F. The US selection method is presented in the HEC-23 manual [107, 108] and NCHRP 593 report [109] and uses the Selection Index approach. The UK selection method is presented in the CIRIA manual [4] and suggests empiric (experience based) approach.

For the purpose of the design of the rip-rap, tool “raplab” was developed. The “raplab” code is shown in Annex G.

3.2.5 Monitoring and Prediction Module

Should immediate maintenance and repair works be proven as too expensive and not within the fiscal year budget plans, a setting-up of a monitoring system with the support of various prediction model(s) can be used as a mitigation measure for ensuring safe traffic over the bridge. The sensors, or network of sensors and models are the basis of the monitoring and prediction module. Falls in the price of the sensors and the progression of ICT technology development have enabled inclusion of monitoring systems straight from the design and construction stages of a bridge's lifespan. Figueiredo et al. [98] estimate that the initial investment cost of a SHM system, for new bridges, to be around 0.5% of the total bridge construction cost. Queensferry Crossing in Scotland [100] is an example where a sophisticated monitoring system was designed and planned from construction to commissioning phase.

Monitoring of the bridge can be an alternative to repair works in case that the cost of design and deployment of monitoring system over cost of repair works can be justified.

3.2.5.1 Structural Health Monitoring (SHM)

As described by Figueiredo et al. [98], *the basic idea of SHM is to build up a system similar to the human nervous system, where the brain (computer) processes the information and determines actions (maintenance activities), and the nerves (sensors) feel the pain (damage).*

The SHM can be periodic or continuous. The continuous SHM relies on principle where if any sensors read any values that are above, or below, pre-designed thresholds, warning is issued. Figueiredo et al. [98] provides good description of SHM techniques. Most SHM techniques traditionally have a strong focus on the bridge structure, most often using accelerometers for measuring vibrations and displacements. The SHM of bridge scour is usually often overlooked. Prendergast and Gavin [110] in 2014 give a review of bridge scour monitoring techniques. In 2015, Michalis et al. [111] proposed the scour probe which estimates scour depth based on a series sensors that measure soil electromagnetic properties. The method yet needs to be tested in a field environment. In 2016, Prendergast [112] proposes estimating scour around bridge foundations using vibration measurements using series of accelerometers. Prendergast et al. [113] were assessing the bridge performance under flooding and seismic actions in 2018.

3.2.5.2 Environmental monitoring as part of Environmental Prediction Module

The importance of environmental monitoring has become more recognised in recent times, especially for bridges over water. The environmental monitoring consists of real time monitoring of water levels, tides and flow velocities at the bridge, and temperature, humidity, soil moisture and rainfall observations on the catchment on which the bridge is located. This data is used for issuing now-casted warnings based on water levels and tides and as an input for modelling of the rainfall runoff process at the catchments. This approach is a direct measure for adapting the infrastructure for future unpredictable climate.

The purpose of the environmental prediction module, or forecasting, is to enable easier planning of any activities around the bridge over the water, from the bridge's construction through to end of service. Many of the bridge components are most vulnerable during the construction phase and civil engineering works in water, especially flowing water, require extra measures in order to ensure safety and quality of works. The flood forecasting system, whose pilot-case implementation is described in Chapter 8, can assist in planning of works around the bridge during its construction, for planning of the inspections and maintenance. Furthermore, in addition to scour inspection, the environmental prediction module can be used to predict the extent of scour giving a measurable value of scour extent relative to bridge foundations, as shown in section 8.3.4.

The implementation of a prediction module using environmental monitoring is elaborated in Chapter 8.

3.2.6 Financial management module

This component processes all of the information on costs from the past and present and gives projections for future projects. The financing module assists bridge network management agencies in managing funds by planning and scheduling of works and activities around the bridge, issuing purchase requests, making payments, generating of the financial reports and projections for short term and long term investments plans in a safe, standardised and transparent way.

Funding is a main driver for all activities within the management system. For a public network, the source of financing, from the bridge management point of view is Government. Government allocates the funds and the responsibility of the bridge owners is rational and efficient spending of the allocated funds in order to ensure maximum safety of the traffic. The production of the financial projections, which include the current state of the infrastructure and future projected infrastructure deterioration, is important in order to enable bridge managers to notify the Government in a timely manner of the future financial requirements of BMS.

Bridge inspections and prioritisation of the bridges is a proven technique to highlight the need for funds allocation. In Ireland the most of the funds for road infrastructures is allocated to paving and resurfacing of the roads, whereas the allocated funds are served just to maintain the current state and prevent further deterioration of the infrastructure and not to improve the safety of the traffic.

Besides planning, part of the financing module is focused on a safe and transparent purchasing system, involving a procurement procedure, issuing requests for purchase, storing quotations, issuing purchase orders, approving and conducting of the payments, and even managing of Human Resources (HR) and payroll.

The rail and road authorities around the world are facing challenges related to bridge management as ageing of the infrastructure results in escalating maintenance requirements of large infrastructure assets. In addition to the need for retaining and improvement of the current state of infrastructure, the need for expanding existing infrastructure is essential. The main restriction on maintaining existing and future infrastructure is the amount of funding available. According to the author's discussions with TII, the funds for bridge management are low and are becoming tighter. Dromey et al.'s [6] identified cost of €24.4 million for rehabilitation of 1400 bridge stock in Co. Cork in Ireland. Most of the funds are allocated for bridge surfaces. Thus there is an increasing need for the BMS that would deliver the most cost-effective way of maintaining of the existing and future infrastructure. Dromey et al. recommend [6] development of an integrated bridge prioritisation index as a decision making aid for targeted allocation of resources for the rehabilitation of bridges on a regional road network in Ireland. Dromey et al. [6] recommend building up the research on the Valenzuela et al. [114] model. The fact that the cost of collection of input data can be significantly higher than maintaining overall BMS is slowing the introduction of new systems. A sophisticated and automated Decision Support System which is capable of storing and processing large amount of the data from bridge inspections, is the core element that can improve the effectiveness of the BMS.

3.2.7 Decision support system

A conventional BMS uses a crisis management approach. This approach is based on repair and not on the maintenance approach, which is, as already indicated, more expensive and consequently inadequate. There is a need for an intelligent decision support system, which would act as an artificial network (brain), that relies on the bridge inspections, sensor network, model results, environmental forecast, resources, financial planning and financial forecast. The modern DSS merges data from all the above models by collecting all the data from the inspections, monitoring network, models and prediction modules and processes the data using automated mathematically-based algorithms and provides information and recommendation(s) to support decisions on financial planning, scheduling of the repair works, further studies and investigations or end of bridge service, hence bridge closure and replacement.

With advances in computational power allowing for fast analysis of large amounts of data, often referred to as big data analysis, decision support systems (DSS) offer a proficient mathematically-based method for aiding in complex decisions. The decision-making herein relates to the classification of bridge condition and effective planning of maintenance where necessary.

A state-of the art DSS can overcome the main identified problems of a bridge management and provide a service for conducting smart standardised inspections in which judgement is broken down into a series of smaller decisions; it can automate and speed up the data acquisition and report writing; monitor the conditions at the bridge and the state of the bridge in the real time; record history of the bridge for future staff and preserve corporate memory.

In order to be the most efficient, a desirable DSS would also give recommendations and instructions as well as issuing warnings. An example of the DSS for scour assessment can be seen for CAESAR [90] system, where recommendations and outputs from CAESAR were compared to the recommendations from Engineer personnel.

3.2.8 Software and Web-based self-informing system

An online, user login secured web-interface bridge management system era is here. With existing ICT technologies, it is possible to access the data from the database and any sensors from phones, tablets or PC's regardless of the location of the bridge management and administration staff. Engineers or technicians may access the system in the office, in the field or at home. As part of this PhD work, the Author had an opportunity to be involved in the BRIDGE SMS EU FP7 project, which is a showcase of the state-of-the-art Bridge Management system (www.bridgesms.eu). A web-interface of the Bridge SMS platform is shown in Figure 3.2.

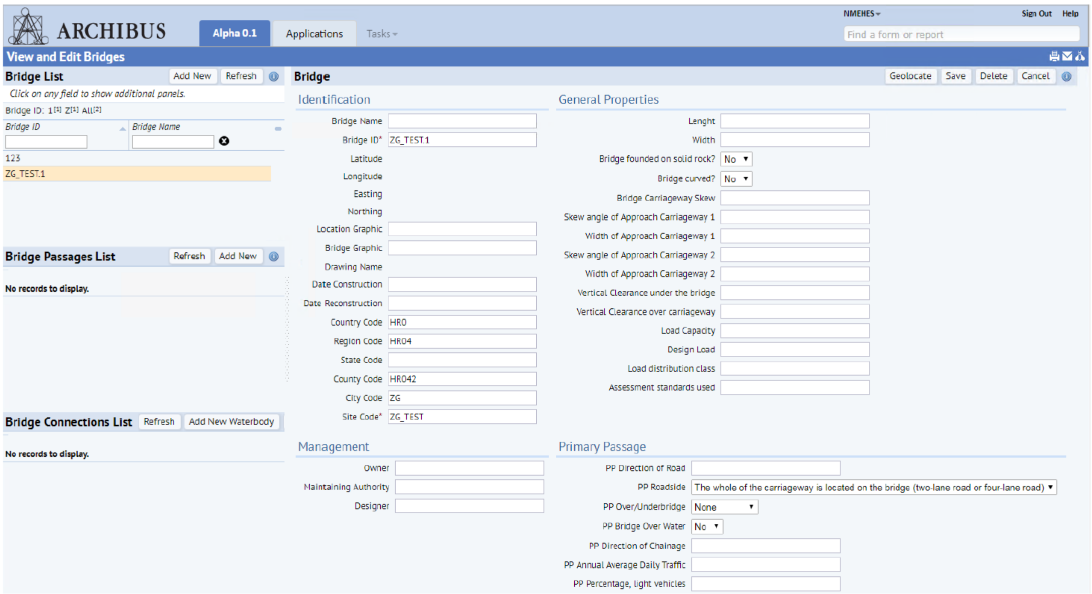


Figure 3.2 Bridge SMS BMS web-interface

3.2.8.1 Software for acquisition of inventory data and bridge inspection process

The bridge inspection process involves planning and scheduling of the inspection(s), performing inspection(s), and reporting. Planning and scheduling of the inspections depends on the previous inspections and resources within the managing agencies. Complicated processes such as bridge inspections rely on the fulfilment of many different specifications or regulations, and it can be excessively time-consuming to ensure that each standard is met during every inspection. A further drawback to complicated inspections is the potential for human error: each additional component requiring inspection creates an added risk that more vulnerable components are overlooked. In order to reduce the length of time taken for inspections and increase the effectiveness of each inspection or maintenance task, informed decisions must take into account both objective and subjective information.

The classical "paper" approach of conducting inspection and reporting is very time consuming. Stepping forward from the "paper" approach is to improve and standardise the process and to include available ICT technologies in the process.

A proposed state-of the art bridge inspection process using ICT technology is shown in Figure 3.3. The figure illustrates how the task for bridge inspection is issued from the Bridge Management platform. The inspection tablet or mobile phone with the installed bridge inspection application needs to be synchronised with the Platform via Wi-Fi connection., then an un-finalised bridge inspection is located on the tablet. The bridge inspector has a tablet with the list of the bridges that he needs to inspect. Based on his location he chooses the closest bridge to inspect. Note that at this point bridge inspection can be conducted in an offline mode as it does not require internet connection when on site. All data such as photographs, component states, quantified values, comments and recommendations are fed into the bridge inspection application on the tablet. After completion of the bridge inspection, when internet connection is available, synchronisation with the Platform can be initiated. Upon completion of synchronisation all data from finished information are located in the bridge management Platform. After the supervisor approves the inspection, all users of the platform can automatically generate a report from the bridge inspection.

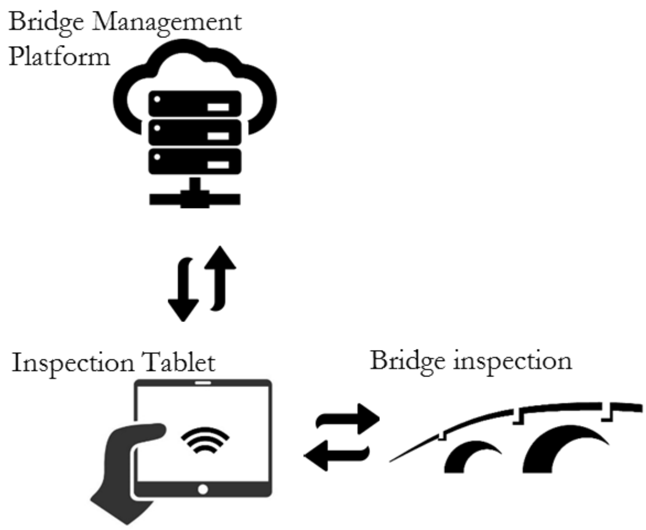


Figure 3.3 Bridge Inspection process using ICT technology and Integration with a BMS database.

3.2.8.2 Methods and software in Bridge Management DSS

3.2.8.2.1 *Multi-criteria decision making (MCDM)*

Multi-criteria decision analysis (MCDA) is a method that can be used in Decision Support Systems (DSS) to decide on and rank factors affecting inspections and structure failure [115]. For multi-dimension DSS problems with numerous criteria and sub criteria, several methods exist that can take into account linguistic descriptions of maintenance requirements based on varied input criteria and output a single value or weighting defining which action to take.

Several methods are introduced here but, whilst the process for each method varies, the overall framework of MCDM is universally applicable and follows the general structure below:

- The MCDM framework scores or ranks the performance of alternative decision options against multiple criteria, which can be either objective or subjective, depending on the type of inputs and outputs required [115].
- All MCDM approaches share some common mathematical elements: the alternatives are ranked and compared depending on the input criterion, where criterion can also be individually weighted depending on sub criteria. In all methods, alternatives and criteria can be given priority weights, which are then multiplied together to produce a total score [116].

Although each approach shares the overall goal of assigning numerical values to complicated decisions, the approach differs significantly in the details of how criteria values are assigned and combined. The processes have different information or knowledge requirements, and the calculated scores have different mathematical properties and, thus, slightly different values and meanings. Table 3.2 lists some examples of MCDM methods usage for varying fields of interest.

Table 3.2 Percentage distribution of MCDM methods per application area [117]

Table 4. Percentage distribution of MCDM methods per application area.

Application area	WSM	AHP	ELECTRE	PROMETHEE	CP	TOPSIS	VIKOR	Combined methods	Total
Water resources systems (%)	4.4	7.4	4.4	13.2	11.8	1.5	4.4	52.9	100
Water and wastewater main (%)	3.8	28.3	15.1	13.2	7.5	1.9	1.9	28.3	100
Transportation (%)	3.6	53.6	8.9	1.8	0	7.1	0	25.0	100
Bridges (%)	0	12.1	0	0	0	0	0	87.9	100
Buildings (%)	3.0	15.2	6.1	3.0	3.0	15.2	6.1	48.5	100
Underground infrastructure (%)	9.1	36.4	9.1	9.1	9.1	0.0	0	27.3	100
Others (%)	4.8	28.6	0	0	0	23.8	0	42.9	100

In the following sections, three alternative decision support methods are introduced: one method is outlined for a single criteria decision and two multiple-criteria decision analysis methods are discussed in greater detail with an examples of a DSS for bridge maintenance decisions.

3.2.8.2.2 *Weighted sum (WSM) and weighted product models (WPM)*

First alternative to Multi-criteria-decision-making (MCDM) is use of Weighted Sum Models (WSM) or Weighted Product Models (WPM). This type of decision system is best for single-dimensional models [118], where it is likely the most commonly used method. For both WSM and WPM:

- The optimal alternative is defined as the one that corresponds to the 'best' value (maximum for all-benefit-type criteria and minimum for cost-type criteria) of the weighted sum [119]
- To apply correctly, all the criteria should be cost type or all-benefit type.
- The difference between WSM and WPM is that weighted parameters are multiplied instead of summed [118].

Advantages:

- WSM and WPM methods are easy to use and understand.

- It is a well-proven technique, applicable when exact and total information is collected, providing good performance when compared with more sophisticated methods [120].

Limitations:

- Normalisation is required to solve multi-dimensional problems, which can be considered a weakness of the methods.
- Because this method works best for single-dimension decisions, it is not the most suitable for bridge maintenance decisions, which must take into account many complex variables.

A more complete discussion follows of methods better suited to multi-criteria decision analysis. The general scour inspection method, proposed in section 6.2 of bridge inspection module, uses Weighted Sum Model approach.

3.2.8.2.3 *Fuzzy Logic*

Traditionally, Boolean logic allows for each criterion to be given a state of either 'true' (or 1) or 'false' (0). However, these fixed-value judgements do not allow for interval distinction between the two extreme values, which is valuable when many different criteria and decisions are possible. The use of fuzzy logic allows for the traditional Boolean logic set to be 'fuzzified' where the 'true' and 'false' values still apply at either end, but in-between these values are a spectrum of possible ratings.

The inputs to the system, termed 'antecedents', are factors that will have the greatest influence on the maintenance decision. The number of antecedents can differ considerably depending on information available and desired complexity of the system. One must be cautious when setting up a system to ensure that the antecedents can be measured and described clearly. For highly complex systems, it can be tempting to include all available information but the larger the input data set, the more possibility for unintended uncertainties. Therefore, it might be best practice to focus on a smaller number of antecedents that can be well defined.

In this case, the DSS using fuzzy logic is executed following the collection of bridge inventory and scour inspection. Upon completion of these steps, the overall bridge structure is given a condition rating and the scour level given a separate rating. Other valuable information is road use, which will help determine the priority of bridge maintenance where the roads can be national, regional or local. The preceding inputs are considered static and maintain a single value throughout the process until the next scheduled inspection. Finally, a dynamic input can also applied here from the flood warning system, which depends on forecasted rainfall and hydrological mapping of the area.

Once the antecedents have been decided, each antecedent is given a 'true-false' set, defined as the "universe of discourse" (abbreviated as universe from here forward). Following definition of the universe, the set is fuzzified where intermediary stages of the set are given linguistic definitions. The output of the system will be one of five fuzzified actions to take depending on information from the static antecedents and dynamic flood data. By only focusing on three static inputs, the system is greatly simplified whilst still taking into account the complex individual factors making up the bridge condition or scour rating. Inclusion of road type is useful for cost-benefit analysis of maintenance decisions. A detailed scour inspection method, proposed in section 6.3 of bridge inspection module, uses a Fuzzy-Logic approach.

Once the sets have been decided and fuzzified, a set of rules are applied to determine the output.

Advantages:

- Extends the use of Boolean logic enabling solution to complex problems
- Enables "intelligent" decision making resembling to human decision making.
- Enables decision when problem(s) cannot be described in precise and discrete terms. Accommodates human and system uncertainty in bridge evaluation [121]

Limitations:

- It is challenging to develop fuzzy rules and membership functions
- a new system with large number of input variables
- The formulation of Fuzzy logic rules is based on expert knowledge
- Challenging to include all available information for larger datasets - more possibility for unintended uncertainties
- Requires verification

3.2.8.2.4 *Analytical hierarchy process (AHP)*

Analytical hierarchy process (AHP), introduced by Saaty in 2008 [122], is a mathematical method for reducing the length of time it takes and improving the final result multi-criteria-based decisions. AHP has the capability of simplifying complex problems by de-constructing the overall problem or goal into a hierarchical system where the top level corresponds to the overall goal, and lower levels concern the criteria (and sub criteria) involved in the final goal as well as alternatives that can be compared for achieving the stated goal [123]. AHP relies on pairwise comparisons between options to improve the weighting given (priority scales) to the different criteria, sub criteria and alternatives in the decision making process [122]. Through quantitative analysis methods, the thinking process when making decisions can be standardised. The procedure for using the AHP can be summarized as [122]:

1. Model the problem as a hierarchy containing the decision goal, the alternatives for reaching it, and the criteria for evaluating the alternatives.
2. Establish priorities among the elements of the hierarchy by making a series of judgments based on pairwise comparisons of the elements. For example, when comparing potential purchases of commercial real estate, the investors might say they prefer location over price and price over timing.
3. Synthesize these judgments to yield a set of overall priorities for the hierarchy. This would combine the investors' judgments about location, price and timing for properties A, B, C, and D into overall priorities for each property.
4. Check the consistency of the judgments.
5. Come to a final decision based on the results of this process.

Advantages:

- Applicable when exact and total parameters are collected.
- Decision problem can be fragmented into its smallest elements, making evidence of each criterion applied.
- Applicable for either single or multiple problems, since it incorporates both qualitative and quantitative criteria.
- Calculation of consistency ratio to assure decision-makers.

Limitations:

- Loss of information can occur due to potential compensation between good scores on some criteria and bad scores on other criteria.
- Implementation is quite inconvenient due to complexity.
- Complex computation is required.

Due to the expanding use of AHP, several software packages: ExpertChoice®, TransparentChoice, SuperDecisions, described below, exist to execute the entire AHP process. Use of commercial software ExpertChoice® would be recommended when resources allow. Open source options exist including the TransparentChoice software and the free SuperDecisions software. Software packages built for AHP include sections for users to fill in factors and sub factors (called criteria and sub criteria within the AHP method) and the alternatives (i.e. separate bridge sections).

3.3 Conclusions on BMS

3.3.1 An assessment of the existing systems and their shortcomings

Historically, the majority of bridge management systems focus mainly on structural issues [1] without adequate emphasis on bridge scour risk. Further, the IABMAS report [1] highlighted that while the BMS are strikingly similar in their overall approach and operation, there was a lack of standardisation, which meant that systems could not be easily adopted by other agencies.

For instance, all BMS, except PONTIS (now AASHTOWare) are used only within the country in which they were developed. The report (p.46) [1] concluded that *"a certain level of standardisation could potentially enhance the exchange of knowledge and experience between managing agents, and improve the usefulness of management systems."*

The DANBRO and recently re-designed EIRSPAN system [96, 97], are advanced bridge management systems both relying on the bridge inspections. However, there is an obvious lack of focus on scour inspection. Only one component reflects the state of the river bed. This means that without detailed instructions and training on how to qualitatively assess the condition of the river bed relative to the safety of the bridge it cannot be considered adequate for assessing scour condition at the bridge. The possibility for error during scour risk assessment due to possible subjectivity of a technician or Engineer is too high.

According to Figueiredo et al. [98] the current bridge inspections and maintenance strategies in Portugal need to be improved. The current SGOA system, similar to DANBRO and EIRSPAN, has proved to be a useful inventory system; however, it needs to be more effective in terms of optimal maintenance program at project level and prioritization of maintenance at network level. Further, there is a need to automate the introduction of information derived from the bridge inspections into the BMSs (page 81 of Report [98]). The same conclusion can be made for the DANBRO and EIRSPAN systems where the lack of synchronisation of the data from the bridge inspection with the database is apparent.

BridgeWatch® system provides excellent basis for the implementation of SHM and environmental sensors, however it is still not a complete bridge management system. It does not have a bridge inspection procedure. The integration of the bridge inspections into BridgeWatch® system would significantly improve the overall functionality needed to be considered full BMS.

The Author cannot give more detailed comment on the BaTMan (Sweden), BAUT (Austria), KUBA (Switzerland), and SIB-Bauwerke (Germany) systems as the documents and procedures on the system are not publicly available.

3.3.2 Recommendations and Motivation for the improvements

The core of the state-of-the-art BMS is a centralised and secured Decision Support System, accessible from any part of the globe. In order to work, the system needs all relevant information to recommend an appropriate action. This information can be obtained from sensors and models from prediction module or from the bridge inspection, hence human observation.

Bridge Inspections give an important, human-assessed input into a DSS. In a way, the **Bridge Inspection module is foundation for the DSS as it provides the most useful, and often the most to date information on the bridge state** supported by recommendations. Historically, recommendations from the reports would remain on paper, hence, within the report. Currently, there is an obvious gap between the inspection report and the bridge database in which the recommendations from the report can be "lost" and misplaced. The first step is filling the identified gap by input of the recommendations from the inspection into the database. Follow-up steps involve appropriate research, involving interaction between academia, civil and IT engineers and bridge managers and the development of the DSS capable of generating the recommendations based on the inputs from the inspections.

Knowing that the bridge scour is a main cause of bridge collapses worldwide and that due to strong focus on structure only [1], there is an obvious gap in the existing bridge inspection procedures; see Chapter 1 and Chapter 2. Neither one procedure identified in the section 3.1, with an exception of CAESAR system [90], gives a clear, fully standardised instructions how to access the bridge scour risk. There is a need for the development a scour inspection module which can be either part of new BMS DSS or that could be implemented into an existing BMS.

Further use of DANBRO system or its variations, EIRSPAN and SGOA systems is recommended due to high level of the development on structural condition assessment and maintenance, but **there is a need for further improvement of these systems**. To overcome their shortcomings, it is recommended to either (1) introduce an additional type of bridge inspection called “Bridge Scour Inspection” or to (2) extend and automate the decision process on describing state of the “river bed” component.

The decision on which of the two options to implement depends on the perspective of the bridge management agencies, on available resources and the level of the training of their staff. For the “perfect” bridge condition assessment, the structural and scour condition should be part of the same process. Although the Author believes that option two, which equalises significance of inspecting the bridge structure, foundations and the river around the bridge, immediate implementation of merging the two approaches is less probable as currently there are no or very few experts which could conduct both types of the inspections as part of a single inspections process. In fact, there is no adequate and standardised type of inspections that would follow this approach. Should the bridge management agencies wish to immediately conduct additional scour inspection programme then outsourcing of the bridge scour inspections, at least in the initial phase of the investment, would be reasonable action. In this case an introduction of the new type of the inspection would be a recommended option, as initially there would be no trained staff within the agency to conduct this type of inspection.

Although some methods for scour assessment already exist, they are not a standardised part of an inspection module of any BMS. Most of the existing scour inspection methods are still under research, without a real application and implementation, they do not have a clearly defined procedures, e.g. they not standardised and require involvement of highly trained staff, they often depend on the subjectivity of the inspector(s), or are too complex and require time consuming calculations and expensive testing and investigations.

As the bridge inspection is a key component of the BMS system (it provides the most useful, and often the most to date information on the bridge state), the following chapter (Chapter 4) searches, and examines the existing bridge scour inspections and assessment procedures. Chapter 4 and Chapter 5 will provide an answer if there is a need for the development of a new bridge scour inspection methods.

Chapter 4
Bridge Scour Inspection Procedures

This chapter will give an overview and comment on existing scour inspection methods.

4.1 Existing Bridge Scour inspections

4.1.1 Method A - Colorado

The Colorado method [124] from 1990, (in later sections of this thesis referred as Method A), proposed in USDA Forest Service scour evaluation [125], is a simple and fast methodology with an automated rating system which is based on a sum of parameters (vulnerability score) derived from flow charts (see Annex H). A single bridge and watercourse component are evaluated by a separate flow chart. The method is based on point summation, first inside each flow chart and then by summarizing each flow chart's results. The result of summation of flow charts is the Vulnerability Ranking Score (VRS). A higher total value of flow chart summation (VRS) represents greater scour risk. A bridge with at least one pier would consist of 4 flow charts:

1. General vulnerability - Global and constriction scour (maximum 23 pts)
2. Left abutment (maximum 14 pts)
3. Right abutment (maximum 14 pts)
4. The worst pier (maximum 15 pts)

The maximum Vulnerability Ranking Score gives 66 points. The purpose of the assigned VRS is to provide an indicative comparison between bridges. The absolute value of points summarized to give the VRS for a bridge has no physical meaning. After vulnerability ranking of all bridges, the Priority Ranking Class is assigned to each bridge (Table 4.1).

Table 4.1. Method A - Priority Ranking Class (PRC)

Priority Ranking Class (PRC)	Vulnerability Ranking Score VRS)
High Priority	≥40
Medium Priority	31-39
Low Priority	≤30

4.1.2 Method B1 - Bekić-McKeogh

The Bekić-McKeogh method (referred as Method B in later text) was developed as a standard methodology for bridge scour inspections and subsequent actions for Irish Rail in 2009. The detailed method description and analysis is in [126]. The method uses a staged approach of scour risk assessment, based on various standards: BA 74/06 (since May 2012 BD 97/12) [127, 128], CIRIA [4], USDA Forest Service [125], HEC18 [51] and other relevant documents, and involves three stages of risk assessment Stage 1, Stage 2 and Stage 3.

Stage 1 Assessment (B1) - is an initial screening stage and involves the collection of data regarding the bridge, its foundations and the river and any information on the history of the bridge and any problems experienced. The principal element of Stage 1 is an assessment by the Inspector as to whether the bridge could suffer from scour damage at all, and to identify important hydrological and hydraulic characteristics of the combined interaction between the bridge design and the watercourse. The main aim of Stage 1 is to identify those bridges where the risks are significant and remedial action needs to be taken. If there are features that make the risk of scour endangering the bridge very low, then the analysis need proceed no further. Otherwise, the assessment should proceed to Stage 2 (B2b). The main deliverables of Stage 1 are Priority Rating of the bridge scour potential and recommendations. The Priority Rating is an indication of the relative potential for scour damage and need for further consideration and possible action (see Table 11.19). The detailed description of method B1 is given in Annex I.

Follow-up steps for the bridges which have been ranked with ratings 3 and 4, respective to table above are *Move to Stage 2 – Analysis* or *Stage 3 – Plan of Action (PoA).*

Stage 2 Analysis (B2) - involves a prediction of potential depths of scour adjacent to the bridge, then prioritization of those bridges which may be at some risk, as a function not only of the scour depths but of other parameters including the location and relative importance of the bridge.

Stage 3 Strategy - is a management approach and recommendations for a bridge in the light of the Priority Rating. This includes possible further studies and remedial action that could be taken to alleviate potential problems.

When bridge has unknown foundation depth and river bed material and flow conditions are not quantified the bridge is often recommended to *Move to Stage 2 – Analysis*. For the bridges with highest scour risk potential for which it is obvious that the scour is significant and that bridge stability is under risk, the bridge is ranked as with PR 4 *Immediate action required (PoA)*. In this case an urgent *Stage 3 Strategy* with scour countermeasure design and construction is undertaken. In this case the network operator (End user) assigns a contact person who will coordinate necessary actions for the specific bridge within its organisation and departments.

4.1.2.1 Stage 1 – Qualitative Assessment (B1)

The method is described in detail in Annex I.

4.1.2.2 Stage 2 – Quantitative Assessment (B2)

The bridge scour risk level was assessed by the two methods: BA 74/06 Priority rating (Stage 2 Assessment) [127] and Qualitative Scour risk [129]. If the resulted scour risk levels differ then the worst case will be adopted.

The first approach (see section 4.1.3.2) is a deterministic approach by BA 74/06 standard [127] (in further text referred as Method B2a) for which the bridge foundation depth should be known. In the absence of the foundation information a conservative estimation of foundation depth is made. The output of this approach is a Priority rating (*PR*).

The second approach (see section 4.1.4) is a qualitative scour risk assessment (in further text referred as Method B2b). The scour risk by this method is obtained in the absence of foundation information. Detailed description of Method B2b is shown in section 4.1.4. The risk of failure is estimated from a Risk Matrix defined by public tender [130] by using an estimated failure probability and an estimate of associated property losses. The NCHRP method [129] is used for obtaining the likelihood of a hazardous event and the HYRISK method to obtain the cost of bridge failure. The output of this approach is a qualitative scour risk (*RQ*) in a range from 1 to 40, which is then grouped in the four scour risk levels.

The Scour risk [129], altogether with the BA74/06 Priority rating [127] is a main parameter based on which mitigation measures for the bridge are assigned.

Based on the Qualitative scour risk and the Priority Rating, mitigation measures will be selected in accordance with BA 74/06 standard [127] (see Table 4.3).

Table 4.2. Qualitative risk matrix [130].

Qualitative Risk Matrix

Likelihood of occurrence of hazardous event	**Frequent** The hazard will occur on a regular basis during the life of the asset / process / system / procedure. The hazard will be continually experienced	10	10 Undesirable	20 Intolerable	30 Intolerable	40 Intolerable
	Probable The hazard will occur several times during the life of the asset / process / system / procedure. – The hazard will occur often	7	7 Tolerable	14 Undesirable	21 Intolerable	28 Intolerable
	Occasional The hazard will occur a number of times during the life of the asset / process / system / procedure. – The hazard will occur infrequently	5	5 Tolerable	10 Undesirable	15 Undesirable	20 Intolerable
	Remote The hazard is likely to occur at some time during the life of the asset / process / system / procedure.	4	4 Negligible	8 Tolerable	12 Undesirable	16 Intolerable
	Improbable The hazard is unlikely to occur but possible at some time during the life of the asset / process / system / procedure– the hazard may occur in exceptional circumstances	2	2 Negligible	4 Negligible	6 Tolerable	8 Tolerable
	Incredible The hazard is extremely unlikely to occur during the life of the asset / process / system / procedure	1	1 Negligible	2 Negligible	3 Negligible	4 Negligible
			1	2	3	4
Severity of hazard consequenc	Description		Insignificant	Marginal	Critical	Catastrophic
	Consequence to persons		Possible minor injury	Minor injury	Single Fatality or severe injury	Fatalities or multiple severe injuries
	Property loss and environmental Consequence		€20k	€200k	€2m	€20m

Table 4.3. Scour risk and suggested actions (from BA 74/06 standard).

Priority Rating	*Qualitative scour risk QR*	*Suggested Action*
1	1 Intolerable	Bridge needs further consideration, including possible monitoring and scour protection measures.
2	2 Undesirable	No immediate action required. Possible monitoring and scour protection measures. Regular re-inspections required.
3	3 Tolerable	Bridge inspections and after major floods, should examine for signs of scour and bank erosion. If conditions at bridge change then reassessment should be carried out.
4		
5	4 Negligible	No action required.

4.1.3 Method B2a - UK Highway Agency BD 97/12

The UK Highways Agency BA 74/06 manual [127] from 2006, superseded by BD 97/12 [128] in 2012, is made for use on road bridges crossing watercourses in the UK but could be applied to other types of bridges and, with appropriate modifications and caution, to bridges elsewhere in the world. The manual [128] outlines requirements for the assessment of scour and other hydraulic actions at highway structures crossing or adjacent to waterways. It provides processes to determine the level of risk associated with scour effects. It also includes processes to assess the robustness of structures in a flood, and references to measures for reducing risk. Manual BD 97/12 [128] consists of two levels (Stages in BA 74/06 [127]) for scour risk evaluation. The scour assessment process is shown in Figure 4.1.

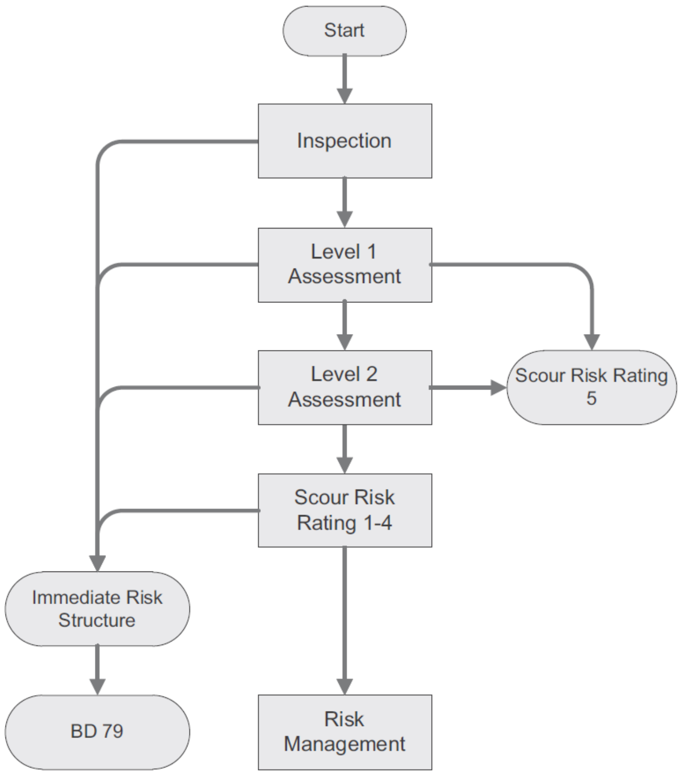


Figure 4.1. Scour assessment process BD 97/12 [128].

4.1.3.1 Level 1 - Assessment

The initial part of the assessment process comprises of an *inspection* and a *Level 1 Assessment*. As described in the BD 97/12 manual [128] The level 1 assessment is a coarse screening method to identify those structures for which the risk of scour damage is tolerably low. It need not involve calculations or numerical analysis, and it may be based to some extent on the judgment of the Assessment Team, considering the information gathered through inspection and from existing records. The outcome of the assessment should be a recommendation either that the assessment should proceed to Level 2 or that the structure should be designated as Scour Risk Rating 5.

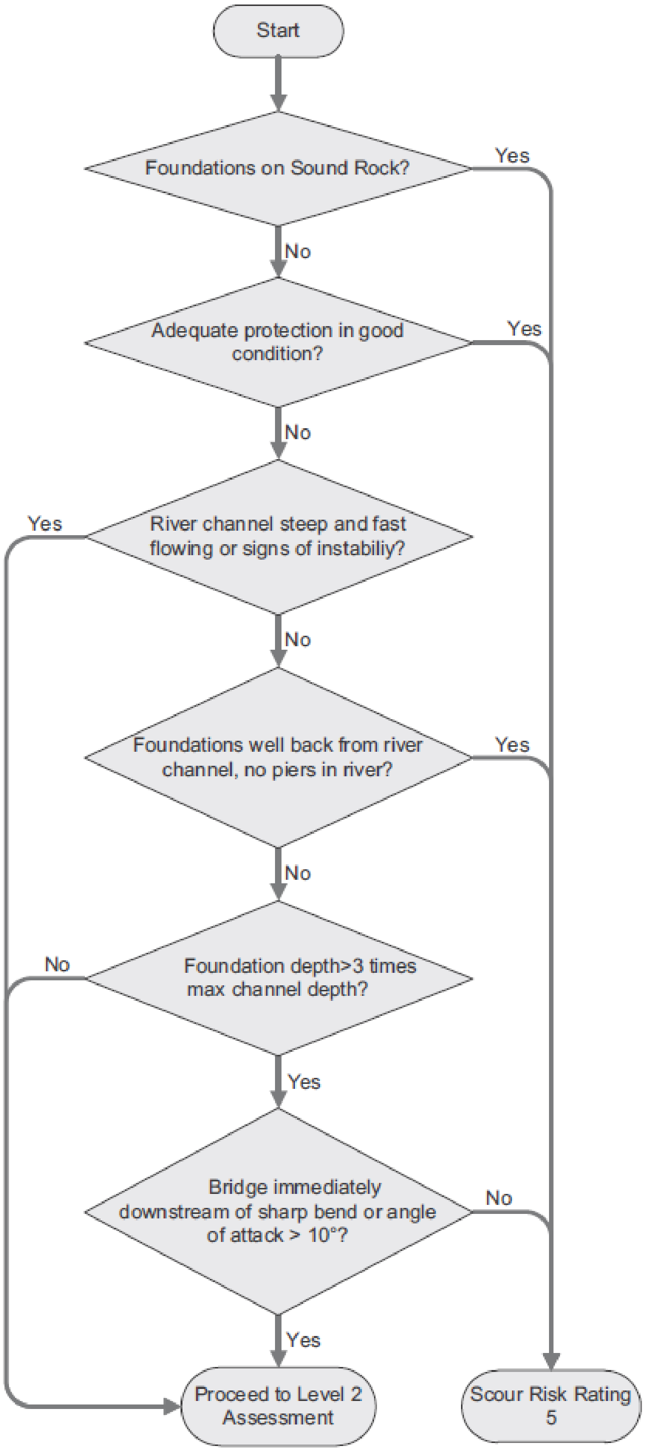


Figure 4.2 Level 1 Assessment Decisions (BD 97/12).

4.1.3.2 Level 2 Assessment (B2a)

The primary purpose of the *Level 2 Assessment* (in later text referred as B2a) is to calculate the relative scour depth corresponding to the Assessment Flow, and to compare this with the foundation level. Based on the Relative Scour Depth (D_R) and the Priority Factor (Pf), see sections 4.1.3.2.1 and 4.1.3.2.2 respectively.

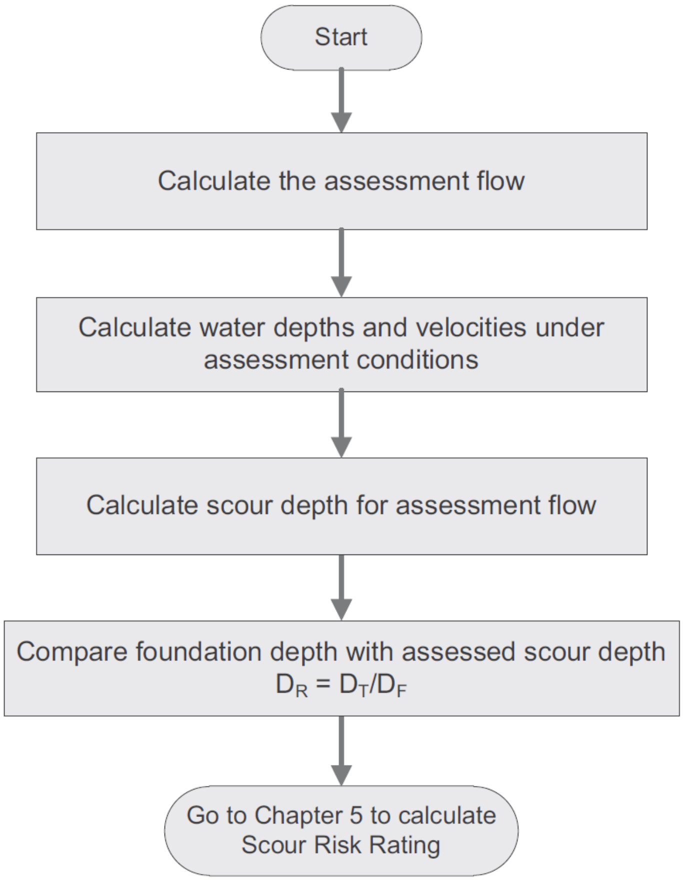


Figure 4.3 Level 2 Assessment procedure (BD 97/12).

The Scour Risk Rating is assessed from Figure 4.4, based on the Priority Factor (Pf) and the Relative Scour Depth (D_R). Figure 4.4 shows five bands which define the risk rating (1 being the highest priority and 5 the lowest). Bridges falling in band 5 have either been eliminated at Stage 1, as having a very low risk of scour damage, or have been assessed in Stage 2 as having a depth of foundation greater than the estimated maximum depth of scour. The Actions for Scour Risk Rating are described in Table 4.4.

Table 4.4. Actions in Response to Scour Risk Rating BD 97/12 [128].

<table>
<tr><th>Scour Risk rating (SRR)</th><th>Actions</th></tr>
<tr><td>1</td><td rowspan="2">Carry out further investigations, determine and if necessary implement appropriate monitoring and scour protection measures as a high priority. Structures with a Risk Rating of 1 to be managed as Immediate Risk Structures in accordance with BD 79.</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td><td rowspan="2">Carry out further investigations, determine and if necessary implement appropriate monitoring and scour protection measures when resources allow and after Risk Rating 1 and 2 structures have been dealt with.
Re-inspections, both as part of regular bridge inspections and after major floods, should examine for signs of scour and bank erosion. If conditions at the bridge change then re-assessment should be carried out.</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td><td>No action required other than routine inspections in accordance with BD 63.</td></tr>
</table>

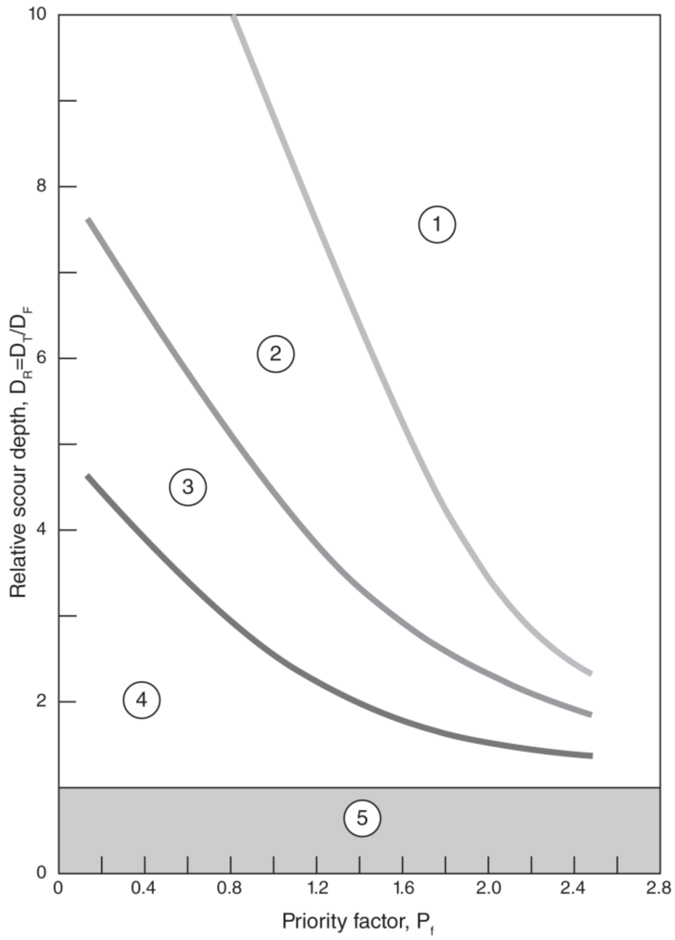


Figure 4.4 Scour Risk Rating [128].

4.1.3.2.1 *Relative scour depth D_R*

On the basis of all design information, hydrological and hydraulic analysis, and calculation of the total depth of scour compared to foundation depths, an assessment of the vulnerability of the bridge to cour damage is made. The assessment is conducted in accordance with the BA 74/06 standard [127]. Total scour depths were calculated for the 200-year flood. For the scour risk assessment, the higher value of total scour depth is used, regardless of whether it is for one of the bridge piers or left or right abutments.

For the bridges with unknown foundation depths the scour risk assessment could not be obtained.

In the case of a bridge that is not eliminated by the Stage 2 Assessment, it is strongly recommended that where the estimates of scour depth are very much greater than the foundation depth but the bridge has no history of problems, possible explanations are investigated. For the bridges with known foundations there are three main cases that may occur:

A. Calculated total scour bed level above the top of the footing
B. Calculated total scour bed level within the limits of the footing
C. Calculated total scour bed level below the bottom of the footing

Relative scour depth D_R (eqn 4.1) is a ratio of the total scour depth D_T and the foundation depth D_F :

$$D_R = D_T \,/\, D_F \text{ [1]} \qquad \text{(eqn 4.1)}$$

A bridge cannot be declared safe if the estimated depth of scour exceeds the depth of the foundation (for $D_R > 1$). However, the calculated scour depth is only an estimate of the potential depth, so if the estimated scour extends below the foundation, it does not necessarily imply that the bridge is at high risk of failure. There are, moreover, many specific reasons why the depth of scour at a bridge may not be as great as the assessment suggests (see BA 74/06 standard [127]). One of the reasons is that methods used for calculating scour depth are conservative and that they over-predict total scour depth.

4.1.3.2.2 *Priority factor P_f*

The Priority factor (eqn 4.2) is a function of foundation type factor F, history of scour problem factor H, foundation factor M and type of river factor T_R. The values used for each factor are shown in Table 4.5. The calculated $\boldsymbol{P_F}$ **is 1.0**.

$$P_F = F \cdot H \cdot M \cdot T_R \text{ [1]} \qquad \text{(eqn 4.2)}$$

Table 4.5. Values for priority factor components.

Foundation type factor F	
For a piled foundation	0.75
For a spread footing	1.0
History of scour problem factor H	
If there is no information on the foundation material or the material is granular (silts, sands, gravels, etc)	1.0
If there is some evidence that the bridge is founded in clay	0.75
If there is strong evidence that the bridge is founded in clay or there is a reasonable possibility of rock under the foundations	0.5
Type of river factor T_R	
If the terrain is mountainous	1.5
If the terrain is upland	1.3
If the terrain is hilly	1.2
If the terrain is lowland or an estuary	1.0

4.1.4 Method B2b - NCHRP Method (Qualitative Scour Risk)

In this approach the main hazards for a bridge are identified and the corresponding risk is described. A *hazardous event* is defined as an event of bridge failure due to scour of the river bed and/or river banks under the bridge.

The scour risk is derived from a qualitative risk matrix from Table 4.2. The *Qualitative scour risk* (*RQ*), in further text Method B2b, is calculated (eqn 4.3) as a product of the *Likelihood of occurrence of hazardous event* (*L*) and the *Severity of hazard consequence* (*S*):

$$RQ = L \times S \qquad \text{(eqn 4.3)}$$

The Qualitative scour risk (*RQ*) is transposed into the four levels of scour risks:

(1) *Negligible risk* for *RQ* = 1 to 4
(2) *Tolerable risk* for *RQ* = 5 to 9
(3) *Undesirable risk* for *RQ* = 10 to 15
(4) *Intolerable risk* for *RQ* > 15

The Likelihood of occurrence *L* and the Severity of hazard consequence *S* are obtained from the HYRISK methodology in the NCHRP report [129]. The NCHRP methodology [129] is in accordance with US National Bridge inventory (NBI). All NBI codes are explained in [131]. Figure 4.5 shows the decision flow from the NCHRP risk assessment tool used for calculation of *Qualitative scour risk* (*RQ*).

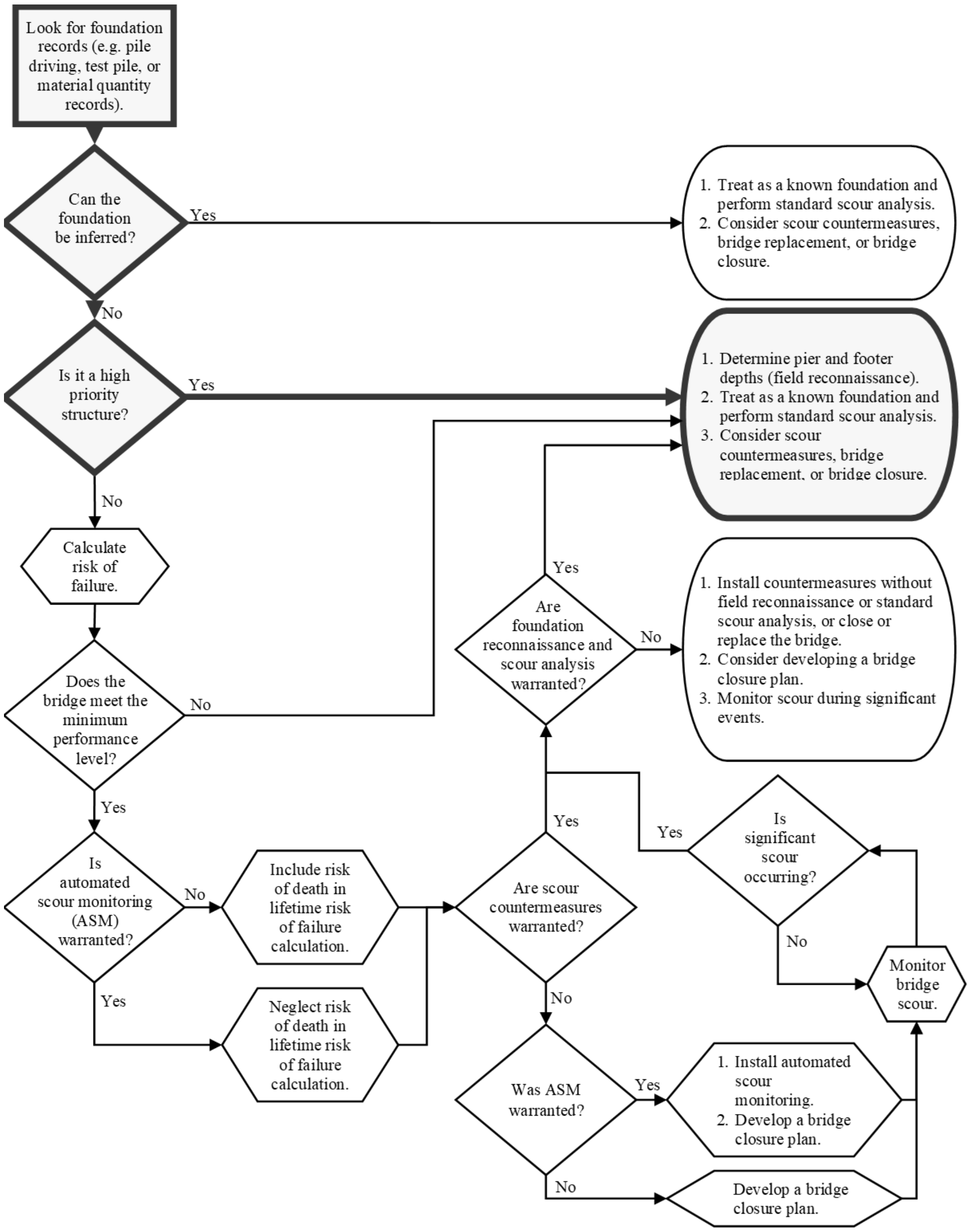


Figure 4.5 Decision flow from NCHRP Risk Assessment tool (w107) [129].

4.1.4.1 Likelihood of occurrence of hazardous event

The Likelihood of occurrence of a hazardous event *L* has six ratings. lhe rating is 10 for Frequent events, and ratings 7, 5, 4, 2 and 1 are for Probable, Occasional, Remote, Improbable and Incredible likelihoods respectively (see Table 4.2).

The Likelihood of occurrence *L* depends on the *Lifetime Risk of Scour Failure* (P_{LT}) which is gained by the NCHRP method [129]. Table 4.6 shows a proposed transformation between the Lifetime risk of scour failure P_{LT} and Likelihood of occurrence of hazardous event *L*. The Lifetime risk is classified in 6 ranks to corresponding to the Likelihood occurrence classification.

Table 4.6. Likelihood of occurrence L relative to Lifetime Risk of Scour Failure P_{LT}.

Lifetime Risk of Scour Failure P_{LT}	*Likelihood of occurrence of hazardous event L*
1	10
0.999 - 0.400	7
0.399 - 0.100	5
0.099 - 0.010	4
0.009 - 0.001	2
<0.001	1

4.1.4.2 Lifetime Risk of Scour Failure P_{LT}

The lifetime risk P_{LT} is defined through (eqn 4.4):

$$P_{LT} = 1 - \left(1 - P_A\right)^{LT} \qquad \text{(eqn 4.4)}$$

where P_A is annual probability of scour failure and *LT* is the provisional life of a bridge[5].

Table 4.7 lists the Annual probability of failure P_A which is a function of (1) Overtopping frequency ratings and (2) Scour vulnerability. Overtopping frequency indicates how often Scour vulnerability is tested. The overtopping frequency and scour vulnerability ratings are obtained from Table 4.8 and Table 4.9 using common NBI data items [131].

[5] For the purpose of calculating the P_{LT} provisional bridge life of 100 years is estimated.

(1) Overtopping frequency is an implied attribute of the (1a) waterway adequacy rating (NBI item 71) and (1b) Functional class (NBI code 26), see Table 4.8. In other words, the overtopping frequency is a measure of a site's likelihood of a scour event, and the HYRISK scour vulnerability is a measure of a bridge's vulnerability to scour failure. Note also that (1b) Functional class (NBI code 26) is in accordance with the US national road network. The Irish Rail lines will be equalized with NBI 26 code in three main groups:

A. Iarnród Éireann rail lines with high frequency (codes 01 and 11)
B. Iarnród Éireann rail lines with medium frequency (codes 02,06,07,12,14,16 and 17)
C. Closed lines (codes 08,09 and 18 in Table 4.8)

(2) Scour vulnerability is a function of (2a) substructure condition (NBI item 60) and (2b) channel protection (NBI item 61) ratings (Table 4.9). The (2a) substructure condition code (NBI item 60) rates the general condition of a bridge's foundation, which should include a qualitative evaluation of how much scour – if any – has been observed at the bridge. Likewise, the (2b) channel and channel protection condition code (NBI item 61) is a qualitative measure of the observed stability of the stream (related to long-term aggradation or degradation)

Table 4.7. NBI Annual probability of scour failure P_A.

Scour Vulnerability	**Overtopping Frequency**			
(from Table 14)	*Remote (R)*	*Slight (S)*	*Occasional (O)*	*Frequent (F)*
(0) Failed	1	1	1	1
(1) Imminent failure	0.01	0.01	0.01	0.01
(2) Critical scour	0.005	0.006	0.008	0.009
(3) Serious scour	0.0011	0.0013	0.0016	0.002
(4) Advanced scour	0.0004	0.0005	0.0006	0.0007
(5) Minor scour	0.000007	0.000008	0.00004	0.00007
(6) Minor deterioration	0.00018	0.00025	0.0004	0.0005
(7) Good condition	0.00018	0.00025	0.0004	0.0005
(8) Very good condition	0.000004	0.000005	0.00002	0.00004
(9) Excellent condition	0.0000025	0.000003	0.000004	0.000007

Table 4.8. Bridge Overtopping Frequency versus NBI Items 26 and 71.

	Functional Class: *(NBI Item 26 Code)*	**Waterway Adequacy** *(NBI Item 71 Code)* (0)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(N)
A	*Principal Arterials, Interstates (01, 11)*	Bridge Closed	Unused	O	O	O	O	S	S	S	R	N
B	*Freeways, Expressways (12)* *Other Principal Arterials (02, 14)* *Minor Arterials (06, 16)* *Major Collectors (07, 17)*	Bridge Closed	Unused	F	O	O	O	S	S	S	R	N
C	*Minor Collectors (08)* *Locals (09, 19)*	Bridge Closed	Unused	F	F	O	O	O	S	S	R	N

Key: *N = Never; R = Remote (T > 100 yr); S = Slight (T = 11–100 yr);*
O = Occasional (T = 3–10 yr); F = Frequent (T < 3 yr)

Table 4.9. Scour Vulnerability versus NBI Items 60 and 61.

Channel Protection *(NBI Item 61 Code)*	**Substructure Condition** *(NBI Item 60 Code)* *(0) Failed*	*(1) Imminent Failure*	*(2) Critical Condition*	*(3) Serious Condition*	*(4) Poor Condition*	*(5) Fair Condition*	*(6) Satisfactory condition*	*(7) Good Condition*	*(8) Very Good Condition*	*(9) Excellent Condition*	*(N) Not Applicable*
(0) Failure	0	0	0	0	0	0	0	0	0	0	0
(1) Failure	0	1	1	1	1	1	1	1	1	1	N
(2) Near Collapse	0	1	2	2	2	2	2	2	2	2	N
(3) Channel Migration	0	1	2	2	3	4	4	4	4	4	N
(4) Undermined Bank	0	1	2	3	4	4	5	5	6	6	N
(5) Eroded Bank	0	1	2	3	4	5	5	6	7	7	N
(6) Bed Movement	0	1	2	3	4	5	6	6	7	7	N
(7) Minor Drift	0	1	2	3	4	6	6	7	7	8	N
(8) Stable Condition	0	1	2	3	4	6	7	7	8	8	N
(9) No Deficiencies	0	1	2	3	4	7	7	8	8	9	N
(N) Not Over Water	0	1	N	N	N	N	N	N	N	N	N

4.1.4.3 Severity of hazard consequence

The Severity of hazard consequence S has four classes (1) Insignificant severity <€20k, (2) Marginal severity around €200k, (3) Critical severity around €2m and (4) Catastrophic severity >€20m. The Severity of hazard consequence is presented as value of money in €.

HYRISK equation (eqn 4.5) from the NCHRP report [129] calculates the total cost of bridge failure and includes[6]:

- Cost of replacing bridge
- Cost of running vehicles on detour
- Cost of lost wages/revenue on detour

As the Cost of lost life (C_{death}) is not covered in this report, the total cost is of bridge failure is:

$$Cost = C_1 eWL + \left[C_2\left(1-\frac{T}{100}\right)+C_3\frac{T}{100}\right]DAd + \left[C_4 O\left(1-\frac{T}{100}\right)+C_5\frac{T}{100}\right]\frac{DAd}{S} \qquad \text{(eqn 4.5)}$$

where,

$Cost$	-	total cost of bridge failure (€)
C_1	-	unit rebuilding cost from (€/m^2)
e	-	cost multiplier for early replacement based on average daily traffic
W	-	bridge width from NBI item 52 (m)
L	-	bridge length from NBI item 49 (m)
C_2	-	cost of running automobile from (i.e. €0.22/km)
C_3	-	cost of running truck from (€1.02/km)
D	-	detour length (km)
A	-	average daily traffic (ADT) from NBI item 29
d	-	duration of detour based on ADT from (days)
C_4	-	value of time per adult in passenger car (€/h)
O	-	average occupancy rate
T	-	average daily truck traffic (ADTT) form NBI item 109 (10% of ADT)
C_5	-	value of time for truck (€22.01/hr)
S	-	average detour speed (typically 64 km/h)

Although the procedure is developed for the road bridges in the US it will be utilized to assess the property loss in a case of Irish Rail bridge failure due to scour.

[6] Cost of Lost Life (Cd_{eath}) is not included so the number of fatality losses is taken as zero.

4.1.5 Method C - Handbook 47 (BSIS or EX2502) detailed scour assessment procedure

The method was first developed in 1989 for British Rail as a preliminary method for assessing the risk to structures from scour [132]. The UK Network Rail procedure [23, 133] relies on estimation of the all three types of scour (general, constriction and local scour), see Chapter 2. The summation of contraction scour and local scour is named total scour (TS), and is divided by the Foundation Depth (FD) to give a Preliminary Priority (PP), see (eqn 4.6) below:

$$PP = 15 + \ln(TS+FD) \qquad \text{(eqn 4.6)}$$

The PP range for most bridges is between 10 and 20. The Preliminary Priority is adjusted for regime (or river type, TR) and the Foundation Material (FM) to give a Final Priority Rating (PR), see (eqn 4.7) below:

$$PR = 15 + \ln(TS+FD) + TR + FM \qquad \text{(eqn 4.7)}$$

A mountainous catchment with high flood severity is considered “flashy” and TR = 0. A lowland catchment with low flood severity is considered “non-flashy” and TR = -1.

Foundation materials are classified as unknown for which FM = 0, as clay for which FM = -1, and rock for which the whole Priority Rating, PR = 10. The Priority Rating is then classified as in Table 4.10.

Table 4.10 Definition of Network Rail Priority Rating (EX2502)

Priority rating	Category	Priority
>17	1	3 - High
16 - 17	2	3 - High
15 - <16	3	2 - Medium
14 - <15	4	2 - Medium
13 - <14	5	1 -Low
<13	6	1 - Low

4.1.6 Other Methods

4.1.6.1 US HEC-18-20-23

The HEC-18 [51] is manual with guidelines for designing new and replacement bridges to resist scour, evaluating existing bridges for vulnerability to scour, inspecting bridges for scour and improving the state-of-practice of estimating scour at bridges. This manual [51] was developed in the USA for the purpose of USDOT-FHWA (U.S. Department of Transportation - Federal Highway Administration) and it instructs DOT's (Departments of Transportation) in the development of scour risk assessment methodologies. This project started in 1988 which resulted in the production of three documents: HEC-18 [51], HEC-20 [52] and HEC-23 [107, 108]. The FHWA uses its own bridge risk assessment and management system which is described in following documents: [51, 52, 107, 108]. The set of documents [51, 52, 107, 108] are the first and the most comprehensive guidelines for bridge scour assessment and as such they are often a basis for development of modified and simplified manuals and guidelines for bridge scour risk assessment and ranking.

The HEC-18 manual is part of a set of HEC manuals issued by FHWA to provide guidance for the bridge scour and the stream stability analyses. The three manuals in this set are:

- HEC-20 Evaluating Scour at Bridges,
- HEC-18 Stream Stability at Highway Structures,
- HEC-23 Bridge Scour and Stream Instability Countermeasures.

The flow chart in Figure 4.6 describes how the three manuals tie together and implies that the three documents should be used as a set. A comprehensive scour analysis or stability evaluation should be based on information presented in all three documents.

The HEC-20 [52] is focused on stream stability (lateral and/or vertical). The outcome of the HEC-20 assessment is either that bridge is ranked as "Low risk" or that the bridge is recommended for further analysis. The HEC-20 stream stability assessment includes both qualitative and quantitative geomorphic and engineering analysis techniques which help establish the level of analysis necessary to solve the stream instability and scour problem for design of a new bridge, or for the evaluation of an existing bridge that may require rehabilitation or countermeasures. A bridge is ranked as “Low risk” in the case that based on a conducted analysis, a bridge inspection, a geotechnical and a minor hydrological-hydraulic analysis stream can be ranked as stable with sufficient certainty. If the stream is ranked as unstable, further analysis described under HEC-18 document [51] is required.

The "Scour Analysis" portion of the HEC-18 block encompasses a seven-step specific design approach which includes evaluation of the components of total scour. The HEC-18 [51] is quantitative procedure which requires a calculation or measurement of total scour depth at the bridge and comparison of total scour depth with foundation depth. Based on comparison of total scour depth with foundation depth a conclusion is made as to whether the bridge is scour susceptible.

If the HEC-18 [51] procedure analysis confirms that a bridge is scour susceptible (total depth scour is close to or greater than foundation depth), design of scour countermeasures is necessary. A design and construction procedure of scour countermeasures is described in HEC-23 manual [107, 108].

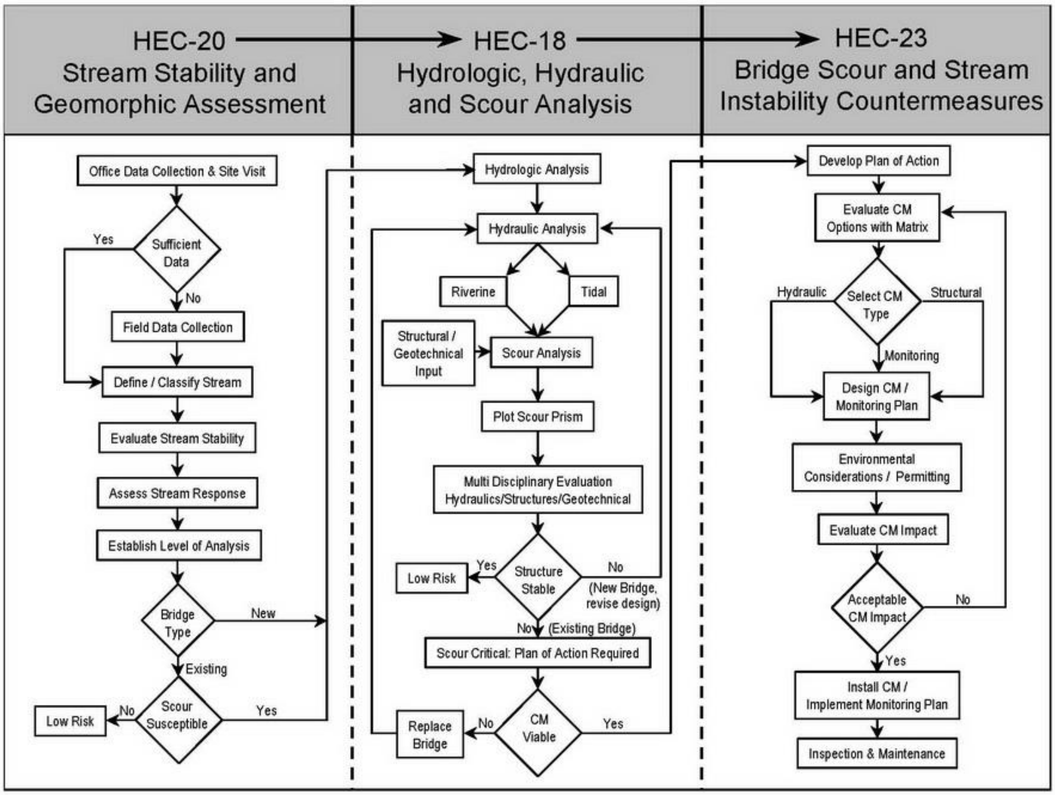


Figure 4.6. Flow chart for scour and stream stability analysis and evaluation [51, 52, 107, 108].

4.1.6.2 Eirspan

The EIRSPAN system Principal Inspection is based on component rating. Ratings from zero to five are assigned to each of the components, with 0 being no damage and 5 being ultimate damage. The list of 14 components [97] of the EIRSPAN system is shown in Table 4.11. The bridge's "Condition rating" is calculated based on the worst-component rating criteria. The first four components do not affect the condition as they do not affect the stability of the structure. However, these first four components are part of the assessment for the purpose of maintenance.

The bridge scour component is part of the 12th component "Riverbed". The "Riverbed" component rating can overpower the rating of any other components in the system as the Condition rating is calculated using the worst-component rating. For example, if rating 5 for the component "Riverbed" is assigned and all other compnents have rating of <5, the "Structure in General" rating, e.g. the bridge Condition rating will be 5. A strength of the system is an online platform with a module for maintenance and damage detection, e.g. cracks guidelines.

The major disadvantage of the system is that there is no clear instruction or guidelines on how to assess the "Riverbed", e.g. a scour component. Scour is described with only two sentences: "Riverbed - This component includes the riverbed immediately upstream and downstream of a bridge as well as the area under the structure. The area around piers and abutments is particularly important, being susceptible to scour damage." [97].

Table 4.11. List of Principal Inspection Components for EIRSPAN [97] and SGOA [98].

No	EIRSPAN Component name	SGOA components
1	Bridge Surface	Pavement
2	Expansion Joints	Expansion Joints
3	Footway / Median	Sidewalks
4	Parapet / Safety Barrier	Safety barrier
5	Embankments / Revetments	Slope
6	Wingwalls / Spandrel Walls / Retaining Walls	Wing walls
7	Abutments	Abutments
8	Piers	Pier / column
9	Bearings	Bearings
10	Deck / Slab	Bridge deck
11	Beams / Girders / Transverse Beams	Cornice
12	Riverbed	Railings
13	Other Elements	Other Components
14	Structure in General	Bridge
15	-	Drainage system
	Condition Rating	Condition Rating

4.1.6.3 SGOA Bridge Inspections - Infrastructures de Portugal

As previously stated in section 3.1.3, the SGOA system defines six different types of inspections: inventory inspection, routine inspection, periodic inspection, special inspection, extra inspection and underwater inspection. The principal inspection does not look the scour issue in detail, rather it focuses on structure of the bridge where 15 components (Table 4.11) are inspected and rated from zero (no damage) to five (ultimate damage) [98]. This inspection procedure is very similar to EIRSPAN and DANBRO.

SGOA system deals with scour issues in much more detail than EIRSPAN and DANBRO systems. - The GOA Manual for Underwater inspection [134] gives detailed description of bridge scour inspection procedure. The Manual [134] divides underwater inspection into: primary, detailed, extra and special underwater inspection.

The description of the scour and hydraulic components is very elaborate and informative. However, the final ranking of the scour component is left to the experience and judgement of the inspector. This approach is a step-forward to a modern BMS ensuring higher safety of the bridge. Although the SGOA system is currently very elaborate , the risk for human error and corporate memory loss (1.1.5) remains high as its system for scour inspection relies on the high level of training and experience of engineers.

Larger bridges over large rivers often require divers and systematisation and standardisation of such inspections is difficult. Improvements of the SGOA system could be done for smaller bridges over watercourses, where walking in the river is possible. A recommended improvement of SGOA is the automation of primary underwater inspection(s). The proposed methodology (Chapter 6) could be used for the automation and further improvement of the SGOA system.

4.2 State of Science on qualitative and quantitative approaches for bridge scour assessment

Quantitative approaches are more detailed, more accurate but require more time and resources than qualitative approaches. Qualitative approaches are simpler, faster and can be used more effectively for initial prioritization.

In 2010, Sathananthan et al. [135] presented a simple qualitative risk-ranking methodology (EIC) for characterising a network of bridges into groups with similar risk levels. This method forms the basis for developing a risk-based inspection regime for a bridge network. A qualitative scoring system uses attributes to rank the bridges in terms of their relative risk by assigning each bridge with an EIC code, where "*E*" is the environmental score (1 for Mild or 2 for Severe), "*I*" is the inspectability score (1 for Easy or 2 for Hard) and "C" is the consequence score (1 for Low or 2 for High). Based on the EIC code, a risk-ranking system is defined, with risk scores ranging from 1.0 (for EIC=111) to 2.0 (for EIC=222). Scour is not considered within this classification. The inclusion of scour as a factor has the potential to be a viable methodology for a rapid initial bridge-ranking strategy. This methodology is demonstrated using the UK's Network Rail bridge stock, whereby a random sample of 18 bridges is ranked according to the proposed method.

The environment factor used in the EIC methodology is comparable to the bridge condition index (SCMI) used in the Network Rail database.

Rapid assessment methods are typically used as the first (zero) step in staged (level) processes of bridge scour assessment. Most qualitative methodologies (Johnson et al. [136-138], Coleman and Malville [5] and Yanmaz et al. [139]) are based on the Johnson et al. [136, 137] approach using 13 indicators of morphologic processes as input data. Yanmaz et al. [139] applied the Johnson et al. [136, 137] methodology on the existing KGM method in Turkey. In 2011, Johnson and Whittington [138] extended Johnson et al. [136, 137] approach by using qualitative descriptions of NBI data [131]. Coleman and Malville [5] use simple observation parameters in order to calculate the scour depth (scour depth is calculated from observed energy slope, sediment transport rate, and median grain size). This approach, due to its simplicity, is also considered as a qualitative scour assessment method.

Quantitative methodologies (Stein et al. [84], Coleman and Malville [5], Briaud et al. [140], Park et al. [85] and Yanmaz and Apaydin [141]) can be generally broken down into two approaches: (1) Stein et al. [84] and the HYRISK approach [129] use NBI data [131] to describe scour risk as a function of probability of failure and bridge failure cost (detailed description is provided in section 4.1.4 above), (2) The exact scour depth in meters is determined. Approach (1) is used in Stein et al. [84] and Yanmaz and Apaydin [141], while approach (2) is used in Coleman and Malville [5], Briaud et al. [140] and Park et al. [85]. Coleman and Malville [5] calculate the scour depth based on hydraulic computations (flow velocity, shear stress, etc.) for each specific case, while Briaud et al. [140] use previously produced scour depth over time (Z) charts for different material types, bridge age and pier and contraction scour parameters. A list of qualitative and quantitative scour risk assessment methods is presented in Table 4.12. From the table it is clear that listed methodologies are tested on a limited number of the bridges and as far as the author is aware, they are not in an operational use.

Table 4.12 Overview of scour risk assessment methodologies.

Approach	Source	Input data: River channel data	Input data: Structural data	Input data: Monetary (cost)	Methodology	Output data forscour	Bridges tested
Qualitative	Johnson et al. (1999)	**13 qualitative and quantitative indicators:** bank soil, bank slope, vegetative bank protection, bank cutting, mass wasting or bank failure, bar development, debris jam, obstructions, deflectors and sediment traps, bed material consolidation and armouring, shear stress, flow angle approach to bridge, distance from meander impact point, channel constriction	n/a	n/a	Summation of 13 qualitative and quantitative indicators multiplied with weight factor (since 2005 it was indicated that weight factors do not need to be applied)	Overall Rating Ranges (Excellent, Good, Fair or Poor) *Note: Assessment of river channel stability only*	8
	Yanmaz et al. (2007)	**13 qualitative and quantitative indicators** (Jones et al. [24, 25]) + Foundation scour of bridge piers and abutments	Deck, Beams, Serviceability components, Drainage, Support, Piers, Border-railing, Abutment, Expansion joint, Ageing of the bridge	n/a	Numerical description of structural and hydraulic evaluation parameters *Note: Renovated Turkey KGM method using methodology proposed by Johnson et al.[24,25]*	Overall Rating Ranges (Excellent, Good, Fair or Poor)	2
	Johnson / Whittington (2011)	**Vulnerability:** **13 Stability assessment rating s** from Johnson [25] **NBI items:** Channel condition (NBI Item 61), Waterway adequacy (NBI Item 71) and Scour (NBI Item 113)	n/a	**Criticality** Summed: detour travel time and NBI items (26, 29, 45, 49 and 51) multiplied by weight factor	Risk is a function of Vulnerability and criticality	Qualitative vulnerability and criticality **Risk-Logic Matrix with Interpretation**	3
Quantitative	Coleman / Malville (2001)	Channel width, radius of curvature, angle of channel confluence, upstream unscoured flow depth, water level rise from low water, channel slope, bed material, critical shear stress, flood flow hydrograph, peak flow rate, duration, discharge intensity	n/a	n/a	**Qualitative** (calculate scour depth from observed energy slope, sediment transport rate and median grain size) **Quantitative** (calculate general, constriction and local scour using empirical formulas)	Total scoured flow depth in [m]	3
	Stein et al. (1999)	**NBI items:** waterway adequacy, scour vulnerability, channel protection. **Hydrological properties of the water stream at the bridge location** (the dimensionless depth describing the height of the water flow)	**NBI items:** typology and topology of the bridge, substructure condition, foundation data	Cost associated with failure	former published as HYRISK Risk = cost of failure x probability of scour failure	annual failure probability The risk of scour failure expressed in currency ($)	n/a
	Briaud et al. BSE 1 (2009)	Maximum observed scour depth at bridge site Z, with its corresponding recorded flow velocity V, age of the bridge, the contraction scour parameters: water depth, soil critical velocity, and contraction ratio, the velocity of the future flood being considered	pier size	n/a	Read future scour from (Z) charts for pier and contraction scour $Z_{fut} = Z_{mo} \times f(V_{fut}/V_{mo})$, Where (Z) is scour depth (Z), (V) is flow velocity, (fut) future, (mo) maximum observed, (f) function based on the material type, age of the bridge, and pier scour and contraction scour	Future scour depth (over time) [m]	n/a
	Park et al. (2012)	Database available from the Water Management Information System web site (www.wamis.go.kr) Stream bed slope, 100-year Design: discharge, water depth, and velocity	Bridge length, Maximum span length, Pile embedded length	**n/a**	Calculate scour vulnerability through dividing (Bearing capacity and safety factor) before scour with (Bearing capacity and safety factor) after scour	Calculated scour depth(m), Bearing capacity reduction after scour, Safety factor after scour, **Scour vulnerability prioritization**	12
	Yanmaz/ Apaydin (2012)	**NBI items** (waterway adequacy, scour vulnerability, channel protection), dimensionless depth describing the height of the water flow, water surface profile and scour depth	**NBI items:** typology and topology of the bridge, substructure condition, foundation data	Cost of bridge failure (direct and indirect), cost of mitigation measures	HYRISK Risk = cost of failure x probability of scour failure	Annual failure probability and an annual risk of scour failure in currency ($)	1

4.3 Conclusions

Existing methods for scour assessment are not fully standardised; their rating systems are unreliable or dependant on expert knowledge and judgement; and are not integrated within the inspection module of any BMS. The summary with advantages and disadvantages of each method is shown in Table 4.13. It can be seen that the methods listed are either too detailed (with high time and cost requirements), in need of standardisation or their scoring systems are inadequate.

Method A shows a clear principle for standardised bridge inspection, however, the scoring system of method A is likely to be inadequate for prioritisation of bridges and is more adequate for inventory purposes, as analysed in [126, 142].

Method B, Bekić-McKeogh, gives promising results [126, 142], however it requires standardisation of the scoring system in order to reduce the time for obtaining the inspection. Method B2a's main disadvantage is the way of calculating estimated scour depth D_T. The calculation of potential scour depth is explained in sections 2.4 and 8.3.4. The lack of described methodology is that empirical formulas to predict scour depth very often give overestimates of theoretical scour when compared to actual scour [143-145]. Mahjoobi et al.'s study [86] showed that model and regression trees are more efficient than the empirical formulas to predict scour depth.

In order to find a hybrid solution between Method A and B, further analysis of methods and development of the rating system for Method B is carried out in Chapter 5.

This thesis will propose a methodology (Chapter 6) that can upgrade the EIRSPAN system and give much more reliable assessment of component 12. The Method (Chapter 6) will increase confidence of inspector(s) by breaking judgement into several components and giving clear guidelines on how to access each of the scour-related components. With addition of a mobile application, the input of the additional information will decrease the time required for inspection and reporting. The UK BD 97/12, NCHRP and Handbook 47 methods will be further analysed in Chapter 7.

Table 4.13 Summary of Bridge Scour inspections

Method	Advantage	Disadvantage
Method A: Colorado, US	Fast and simplified approach	Inadequate rating system [126, 142]. Scour depths is not included.
Method B1: Bekić-McKeogh	Qualitative but very detailed bridge scour inspection. Structured into stages. Good results from scour inspections [126, 142].	Report writing process requires significant amount of time (5 days). Involvement of several persons Need for standardisation of scoring system
Method B2a BD 97/12 (BA 74/06), UK	Detailed, structured into levels	Time consuming, need for hydraulic calculations and quantification. The equations for calculation of scour depth tend to overestimate scour depth. Too detailed for simple bridges.
Method B2b NCHRP	Includes direct and indirect cost of bridge collapse.	The procedure for defining of scour state / bridge condition is not defined. Too detailed for simple bridges. Probability assessment does not include state of the bridge. Does not include a cost of repairs. Currently in place for US bridges.
Method C: Handbook 47	Fast and simple method	Does not take into account bank erosion
US HEC-18-20-23	Detailed Quantitative analysis.	Not standardised, time consuming. Not structured into levels. Too detailed for simple bridges.
EIRSPAN, Ireland	Fast and effective structural inspection.	Inadequate scour inspection.
SGOA, Portugal	Fast and effective structural inspection.	Scour inspection needs standardisation. No scoring system and not automated.

In the following chapter, a comparison and an attempt for upgrading of two existing scour inspection methodologies - **Method A** - Colorado, US and **Method B1** - Bekić-McKeogh will be presented. Chapter 5 will develop a rating system for Method B1 (Table 5.2) which would be similar, but more reliable to the rating system of Method A.

Chapter 5
Bridge Scour Inspection: Selection of the Components and Development of the Rating System

From the methodologies identified in Chapter 4, two methods are selected for further analysis and improvement. These are: Method A - Colorado method (in further text Method A) and Method B1 - Bekić-McKeogh Stage 1, in further text referred as Method B1. The reason that those two methods were selected is that Method A is a very rapid and standardised method, while method B1 is a more detailed method that involves more experts. Also, both methods were applied on the same 100 railway bridges in Ireland. A detailed assessment of the Methods A and B1 (Stage 1) based on the dataset of 100 railway bridges in Ireland was conducted in 2011 [126, 142]. This dataset, with addition of one more bridge (ranked as a culvert) will later be described as Data block 2 (see section 7.3.2). In the work [126] it was shown that Method B1 overpowers the Method A - Colorado method (Method A). Although Method A has an advantage in the time required to conduct the inspection and reporting, the results and weights of some variables (elements) are inadequate, as identified in [126, 142]. Based on its application on a relatively large number of bridges Method B1 was shown to be an appropriate method for bridge scour assessment [126, 142]. **The intention of this chapter is to develop a rating system for Method B1** (Table 5.2) **which would be similar, but more reliable to rating system of Method A.**

Method B1, although qualitative, is a very detailed bridge inspection method which takes approximately 5 days to conduct office screening, inspection, and report writing. Two main restrictions in Method B1 are the need for standardisation of scoring system, which mainly relies on the experience and subjectivity of inspector(s); and the fact that the report writing process requires significant amount of time and the involvement of several persons.

Method A is analysed and compared to Method B1 in this chapter on a dataset for 100 bridges. The goal of this chapter is to propose a new scoring system for Method B1 by identifying the most important variables (elements) which would result in a breakdown of decision into several smaller decisions, based on which a bridge condition rating would be assigned. This could reduce the number of persons involved in the process, reduce the need for highly trained staff and save a significant amount of resources and time.

For the purpose of the identification of the elements for Method A and B1 and the development of the new scoring system for Method B1, a Principal Component Analysis is conducted.

5.1 Principal Component Analysis theoretical background

Principal Component Analysis (PCA) is a multivariate analysis technique [146], the primary purpose of which is to reduce the dimensionality of a data set [147]. A background to the theory is presented here for the sake of completeness; further information on the method can be found in the referenced texts. The desired outcome of PCA analysis is to redefine the input variables as principal components (PC), which are a linear combination of the original variables, but are reduced in number compared to the original set of variables, while preserving most of the information. This is accomplished by highlighting the variables that demonstrate the most variance in the data set. The first principal component Y_1 is defined as (eqn 5.1):

$$Y_1 = \alpha'_1 x = \alpha_{11}x_1 + \alpha_{12}x_2 + \cdots + \alpha_{1p}x_p = \sum_{j=1}^{p} \alpha_{1j}x_j \quad \text{(eqn 5.1)}$$

Where $\alpha'_1 x$ is a linear function of the elements x having maximum variance, and α is a vector of p coefficients α. The sum of the square of the coefficients α_i (eqn 5.2) is equal to unity, and is a better indicator of the influence the coefficient has than the raw value:

$$\sum_{i=1}^{p} \alpha_i^2 = \alpha'\alpha = 1 \quad \text{(eqn 5.2)}$$

The first principal component is the direction along which the data set shows the largest variation [148], and the second component is determined under the constraint of being orthogonal to the first component and to have the largest variance [149]. The second principal component $Y_2 = \alpha'_2 x$ is found in a similar manner to the first principal component, and so on for the subsequent principal components, up to p PCs. It is, however, desired that most of the variance in the data set is accounted for in the PCs $\ll$ p. In order to locate the principal components, it is necessary to determine the covariance matrix Σ of the vector of random variables x. It can then be shown that α_k is an eigenvector of Σ corresponding to its kth largest eigenvalue λ_k [147]. The above can be discussed in matrix terms where a PCA can be conducted through an eigenvalue decomposition (EVD) or a more robust and generalized singular value decomposition (SVD) [150]. For a data matrix X of n observations on p variables measured about their means, see (eqn 5.3):

$$X = ULA' \qquad \text{(eqn 5.3)}$$

Where L is an (r x r) diagonal matrix, and U and A are (n x r) and (p x r) matrices, respectively, with orthonormal columns, and r is the rank of X. It has been observed that the SVD approach to PCA is a computationally efficient and generalised method to determining the PCs.

It has been suggested that PCA should only be conducted on continuous variables that conform to a Gaussian distribution [151], and that its application to discrete data, such as condition ratings of a BMS, is inappropriate. However, so long as inferential techniques that require the assumption of multivariate normality are not invoked, there is no necessity for the variables in the data set to have any associated probability distribution [152]. The practical applications of PCA are many, and include [153]:

- The examination of correlations between variables
- The reduction of the basic dimensions of the variability in the measured data set to the smallest number of meaningful dimensions
- The elimination of variables which contribute relatively little extra information
- The examination of the grouping of individuals in n-dimensional space
- Determination of the objective weighting of measured variables in the construction of meaningful indices
- The allocation of individuals to previously demarcated groups
- The recognition of misidentified individuals
- Orthogonalization of regression calculations

It is often considered wise to use the correlation matrix for a PCA, as the standardized variates are dimensionless and can be more readily compared [147]. However, when the variables are measured in the same units and have a low variance, using the covariance matrix is sometimes appropriate, and can be beneficial when statistical inference is important. In this case, when the condition ratings are already dimensionless, it is not entirely necessary to standardise the variables. A biplot [154] is most often used to visualise the output of a PCA, as it can handle a matrix of a higher rank than two by approximating it as a matrix of rank two. The biplot displays the orthogonal component coefficients for each variable and the principal component scores for each observation. An aspect of the biplot is the plotting of variable vectors, the direction and length of each indicate how each variable contributes to the two principal components in the plot. Further discussion of the method can be seen in depth with the additional references [155-160].

5.2 Method of evaluation

Principal Component Analysis (PCA) was utilised for evaluation of the datasets produced from two bridge scour assessment methods Colorado method (see section 4.1.1) and modified BA 74/06 (see section 4.1.2).

The first step was the development of a single database where all the data from the 200 reports (two methods) was obtained and classified into the variables.

The second step was selection of the relevant variables for the analysis for Method A and B1.

The third step was defining a scoring system for Method B1 using PCA.

The fourth step was utilisation of the PCA on the variables of Method A and B1. Based on the PCA analysis, the significance of the variables for each method was assessed. PCA analysis enabled comparison if the two Methods (A and B1) give equal importance to the same variables. Additionally, regression analysis was made between Ranking Summary and Total scores for the newly developed scoring system of Method B1.
Finally, regression analysis from the two different results (with and without scoring systems) of Method B1 was compared with the Vulnerability Ranking Score (VRS) of Method A.

5.2.1 Description of rankings and comparison of two methods

5.2.1.1 Method A – Colorado rankings

For detailed description of Method A see section 4.1.1 and Annex H.

5.2.1.2 Method B1 – Bekić-McKeogh rankings

For detailed description of Method B1 see section 4.1.2 and Annex I.

In addition, based on the (1) Priority Rating PR and the (2) recommendation for the Years to the next inspection, (3) the Rank Summary (RS) is calculated. This rank (RS) was firstly defined in work [126] and will be used for the comparison. The principle for calculation of Rank Summary (RS) is described below:

- Rank „Insignificant risk“ has a RS of 10 points;
- the RS for bridges ranked as „Low risk“ is calculated based on to the equation (eqn 5.4)):

$$RS = 40 - (5 \cdot yr) \qquad \text{(eqn 5.4)}$$

where *PR* is Priority rating and *yr* are years to the next inspection

- Rank „Move to Stage 2 – Analysis is evaluated with 40 points.
- Rank „Immediate action required (PoA)“ is evaluated with 50 points.

Rank Summary (RS) will be used for the comparison with Vulnerability Ranking Score (VRS) from Method A and a new scoring system in Method B1.

5.2.2 Element Description

Both methods, A and B1, were utilised for the assessment of the same 100 railway bridges. The network of railway bridges in Ireland is one of the oldest in the world. After the Malahide Viaduct collapse in August 2009, the railway authority (Irish Rail) conducted the scour risk assessment on entire network of bridges in the country. Based on 100 reports of the Colorado method and 100 reports of Bekić-McKeogh method a single database was created. The spreadsheet/database is formed in a way that the rows represent bridges and columns represent different Variables extracted from the reports from Method A and B1. In further text a description of the variables used in the PCA analysis will be given. The list of input variables (elements) is shown in Table 5.1. The distribution of the condition ratings for the individual elements for both methods are shown in box plots (Figure 5.1). In this plot, the edges of the box represent the 25th and 75th percentiles of the data, with the central marker showing the median.

For the Method A, the variables are grouped with assigned codes in accordance to four flowcharts from the Colorado method (Annex H). The variables from the General vulnerability use codes from Gv1 to Gv11, the variables from the Left and the Right abutment flow charts use codes from LA1 to LA5, and from RA1 to RA5 respectively, and the variables from the worst pier flow chart use codes from P1 to P5. A significant amount of data generated from 100 bridge inspections using Method B1 (Bekić-McKeogh) needed to be organised into variables whose significance would then be verified using PCA. The resulting Method B1 variables are listed in Table 5.1.

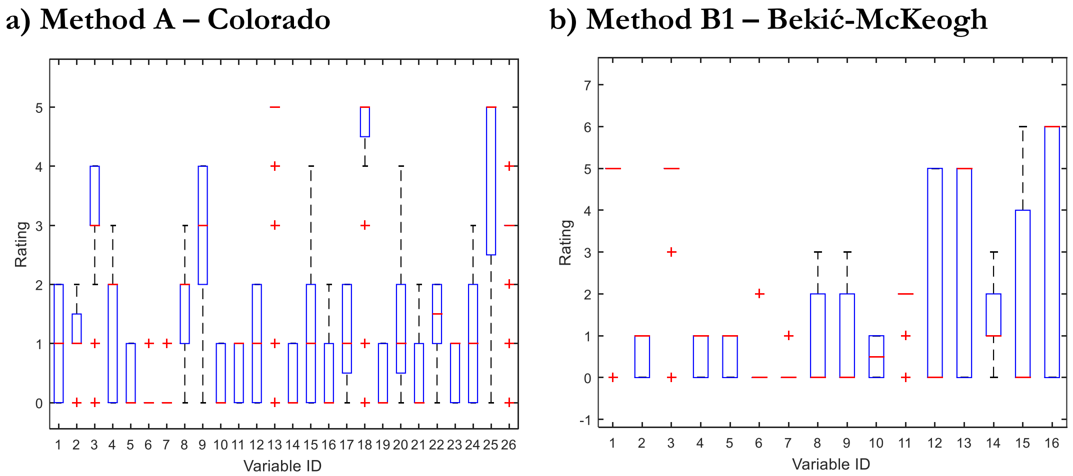


Figure 5.1. Boxplot of the variable ratings for the Method A and B1

Table 5.1. List of elements for Method A and B1 used in PCA.

ID	Code	Variable (element)	ID	Code	Variable (element)
1	Gv1	River Slope/Velocity	1	v1	Scour susceptible bed material
2	Gv2	Channel Bottom	2	v2	Debris accumulation potential
3	Gv3	Channel Bed Material	3	v4	Foundation type
4	Gv4	Channel Configuration	4	v5	Flooding History
5	Gv5	Debris/Ice Problem	5	v6	Inundation flow
6	Gv6	Near River Confluence	6	v7	Low deck / Possible pressure flow
7	Gv7	Effected by Backwater	7	v8	Tidal river
8	Gv8	Historic Scour Depth	8	v9	Local bank erosion downstream
9	Gv9	Historic Maximum Flood Depth	9	v10	Local bank erosion upstream
10	Gv10	Adequate Opening	10	v11	Skew angle
11	Gv11	Overflow/Relief Available	11	v12	Bridge type
12	LA1	LA Scour Countermeasures	12	x22b	General channel stability lateral
13	LA2	LA Foundation	13	x22c	General channel stability vertical
14	LA3	LA Location on River Bend	14	x23	Constriction scour potential
15	LA4	LA Angle of Inclination	15	x24a	Abutment scour
16	LA5	LA Embankment Encroachment	16	x24b	Pier scour
17	RA1	RA Scour Countermeasures			
18	RA2	RA Foundation			
19	RA3	RA Location on River Bend			
20	RA4	RA Angle of Inclination			
21	RA5	RA Embankment Encroachment			
22	P1	Pier Scour Countermeasures			
23	P2	Pier Foundation			
24	P3	Pier Skew Angle			
25	P4	Pier Bottom Below Streambed			
26	P5	Pier Width			

5.2.3 New scoring system for Bekić-McKeogh Method

The new scoring system is developed using PCA. In order to establish a scoring system, the number of variables and their weights (scores) were varied until satisfactory results were achieved. The proposed scoring system is shown in Table 5.2.

Table 5.2. List of Variables used in PCA with their ratings for the scoring system of Method B1.

ID	Code	Variable	Maximum Score	Weight
1	v1	Scour susceptible bed material	5	0.10
2	v2	Debris accumulation potential	1	0.02
3	v4	Foundation type	5	0.10
4	v5	Flooding History	1	0.02
5	v6	Inundation flow	1	0.02
6	v7	Low deck / Possible pressure flow	2	0.04
7	v8	Tidal river	1	0.02
8	v9	Local bank erosion downstream	3	0.06
9	v10	Local bank erosion upstream	3	0.06
10	v11	Skew angle	1	0.02
11	v12	Bridge type	2	0.04
12	x22b	General channel stability lateral	5	0.10
13	x22c	General channel stability vertical	5	0.10
14	x23	Constriction scour potential	3	0.06
15	x24a	Abutment scour	6	0.12
16	x24b	Pier scour	6	0.12
		Total:	50	1.00

A description of the variables and ratings of the developed rating system follows:

(1) **Scour susceptible bed material.** If the river bed material is scour susceptible (sand and fine gravel) the variable Scour susceptible bed material should be rated with 5 points. If the river bed is rock / bridge founded on rock value of 0 should be assigned.

(2) **Debris accumulation potential.** If there is Debris accumulation flow potential, this variable should be rated with 1 point. Otherwise it should be 0.

(3) **Foundation type.** If the bridge foundations are unknown or known and shallow or undermined, variable "1 Foundation type" should be rated with 5 points. If the bridge is founded on rock or the depth of foundations is more than twice of the depth of potential scour, 0 points is assigned. In case that the foundation depth is varying but if it is not shallow and less than twice of depth of potential scour, then 3 points should be assigned.

(4) **Flooding History.** If there is flooding History / scour history at the bridge, 1 point is assigned, otherwise zero (0) points is assigned

(5) **Inundation flow.** This variable is closely related with variable 14, constriction scour potential. If there is inundation flow (overtopping of the floodplains) during the flooding, due to increased constriction scour potential 1 point is assigned to the bridge. If there is no overtopping of the floodplains and no obstruction to the flow at the floodplains 0 points is assigned.

(6) **Low deck / Possible pressure flow.** If the bridge deck is low and if there is possibility of the water levels reaching the bridge deck/soffit level and causing accumulation of floating debris or even pressure flow through the bridge opening (increased flow velocities) 2 points are assigned. If there is no such risk 0 points should be assigned.

(7) **Tidal river.** If the river is tidal due to more complex hydraulics and possible increase of flow velocities due to tide withdrawn 1 point should be assigned to the bridge. If it is not tidal at the bridge 0 points should be assigned.

(8) **Local bank erosion downstream.** If there is erosion of the both banks downstream of the bridge, 3 points should be assigned. If there is erosion of one bank (left or right) 2 points should be assigned. If there is no erosion of the downstream river banks, 0 points should be assigned.

(9) **Local bank erosion upstream.** If there is erosion of the both banks upstream of the bridge, 3 points should be assigned. If there is erosion of one bank (left or right) 2 points should be assigned. If there is no erosion of the upstream river banks, 0 points should be assigned.

(10) **Skew angle.** If the bridge skew angle (flow attack to bridge piers / abutments) is less than $\geq 30^0$, 1 point should be assigned. If the skew angle is between $\geq 10^0$ and $<30^0$, 0.5 points should be assigned. If the skew angle is less than 10^0, 0 points should be assigned.

(11) **Bridge type.** If the Bridge is complex, 2 points should be assigned, if the bridge is simple (single span), 1 point should be assigned, if the bridge is a culvert or with span less than 1.0 m 0 points should be assigned.

(12) **General channel stability lateral.** If the river channel is laterally unstable, 5 points should be assigned. If the river channel is laterally stable, 0 points should be assigned.

(13) **General channel stability vertical.** If the river channel is vertically unstable, 5 points should be assigned. If the river channel is vertically stable, 0 points should be assigned.

(14) **Constriction scour potential.** If the bridge construction obstructs the flow and causes significant alteration of natural flow conditions, 3 points should be assigned. If there is constriction on both floodplains 2 points should be assigned, 2 points should be assigned. If there is constriction on one floodplain, 1 point should be assigned. If there is no constriction, 0 points should be assigned.

(15) **Abutment scour.** If the scour is evident on both abutments, 6 points should be assigned. If the scour is evident on one of the abutments (left or right), 4 points should be assigned. If there is no abutment scour, 0 points should be assigned.

(16) **Pier scour.** If the Pier scour is evident 6 points should be assigned. If there is no Pier scour, 0 points should be assigned.

5.3 Results

5.3.1 Comparison of Method A and B1 rankings

As indicated above, Method A Vulnerability Ranking Score (VRS) is compared to Method B1 Rank Summary (RS) on the sample of 100 railway bridges in Ireland. A comparison of results by two methods showed that assessments deviate for 19 bridges.

Based on three examples in Figure 5.2 (indicated by black filled dots), the discrepancies in the results will be explained.

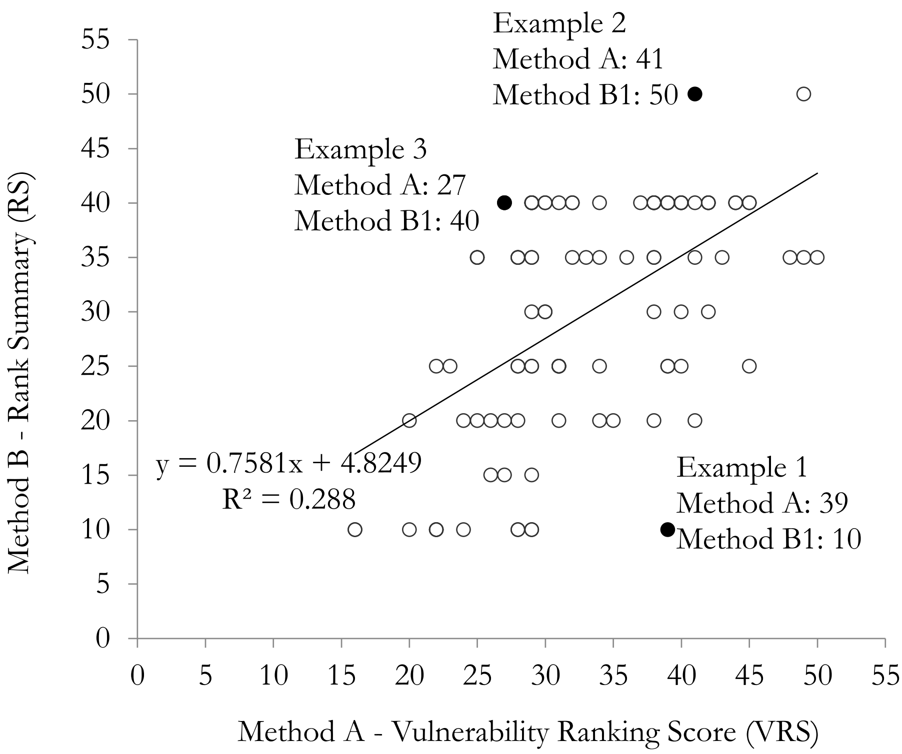


Figure 5.2. Comparison of Method A – Vulnerability Ranking Score (VRS) with Method B1 Ranking Summary for 100 railway bridges in Ireland.

Example 1 shows how Method A can give relatively high VRS score for a bridge that is ranked with insignificant risk in Method B1. Due to the higher number of elements, not necessary their poor condition, of the bridge and the incapability of Method A to observe that bridge is founded on solid rock, a high VRS is assigned. Method B1 for the specific bridge was able to recognise that the bridge is founded on solid rock and that there is no scour risk for the bridge in foreseeable future. The pattern when Method A over predicts VRS for the bridge can be noted on at least 10 bridges from the dataset.

Example 2 shows how, for a bridge highly vulnerable to scour, as indicated in Method B1, Method A does not assign the highest possible VRS (in above example, VRS of 41 is assigned while 66 is maximum).

The third example and the real issue that is noted on at least 9 bridges is when for a highly vulnerable bridge, method A assigns relatively low VRS (score of 27 in the noted example in Figure 5.2).

According to the obtained results and presented examples it can be concluded that the methods which use the Colorado flow charts and Vulnerability Ranking Score are unreliable for evaluation of bridge scour risk on complex terrain and should be carefully utilized in rapid assessments of bridge scour [142]. The Method A (Colorado) should be used only for inventory purposes.

5.3.2 Results from Principal Component Analysis

Principal Component Analysis was applied on a network of 100 railway bridges in Ireland using two methods Method A (Colorado) and Method B1 (Bekić-McKeogh). From 100 bridges, 6 bridges classify as culverts, 17 bridges are simple bridges (single span bridges with simple hydraulic conditions) and 77 bridges classify as complex bridges. This dataset, with the addition of one more bridge (ranked as culvert) will later be described as Data block 2 (see section 7.3.2).

The number of important principal components is visualised with a scree plot of the eigenvalues [161] (see Figure 5.3). This can be a useful tool for determining what components are important and what components can be discarded from the data set. As the eigenvalue in the scree plot drops, the components become less important as they retain less variance than the previous components. There is a progressive decrease in the influence of the higher-order eigenvalues, and it has been determined that the first three components (as an indicative rule, components with an eigenvalue less than the average can be discarded) retain most of the variance for both Methods (A and B1); accounting 67.57% and 58.61% of the data respectively, see Figure 5.3.

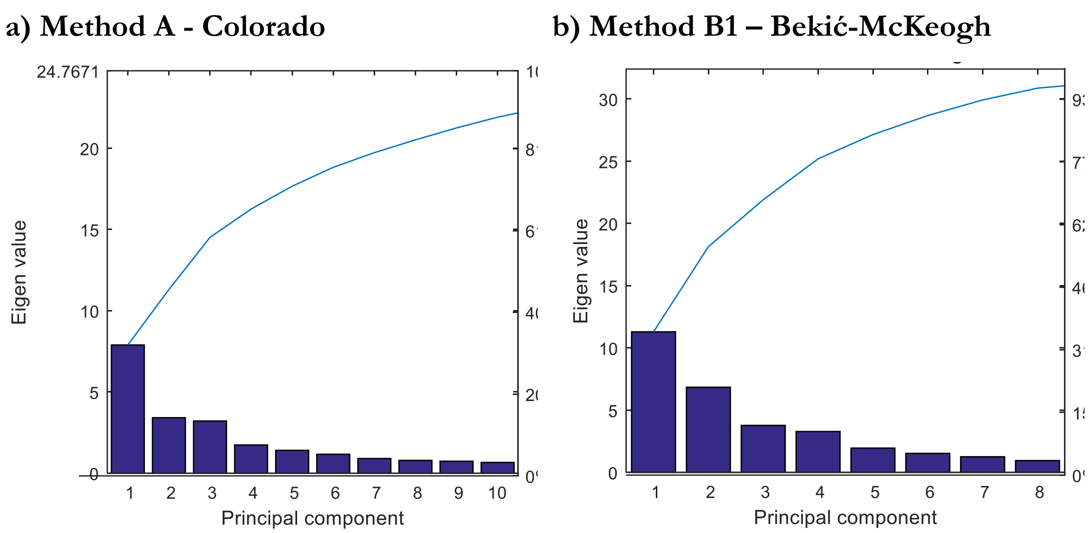


Figure 5.3. Scree plots of Principal components for Method A and B1.

The bar graph (Figure 5.4) shows the degree of significance of the variables. All scores were multiplied by corresponding eigenvalues. Final maximum scores for the variables were selected based on their absolute value, leaving the algebraic sign assigned to the score. The variables with a different algebraic sign indicate that two components behave conversely. For example in Figure 5.4a, Method A suggests that if the Left abutment Foundation has higher value (shallow), the embankment Encroachment would be smaller and vice versa. As this could be a result of a site conditions (smaller river, bridge founded on the rock, etc.) or poor design (bridges with shallow foundations that are prone to scour would usually have scour protection in place), we cannot completely disregard the adequacy of the method (Method A). However, Method A should be looked at more closely. Following the same logic, Figure 5.4b for Method B1, suggests that if the banks are stable (or man-made), the bridge is more prone to a vertical instability. An example of vertical degradation of a river bed with constructed embankments was given in section 2.2 for the river Sava in Zagreb, Croatia.

Table 5.3 and Table 5.4 give detailed descriptions of the Principal Component eigenvalues and percentage variance explained, representation of the variables for principal components with their factor loadings, means and standard errors for Method A and B1 respectively. The tables indicate which variables are best described by each principal component.

For Method A, Principal Component (PC) 1 best describes abutment foundations, scour countermeasures, bridge opening, available openings in the embankments (side culverts / relief), river bend and backwater effect; PC 2 best describes skew angles and abutment inclination angles, historic flooding and scour depths, and river slope; PC 3 describes pier bottom and foundations, embankment encroachment and scour countermeasures, channel bottom and confluences with other rivers. For Method B1, PC 1 best describes pier and abutment scour, lateral channel stability and bank erosion, foundation type, bed material and constriction potential; PC2 best describes vertical channel stability; PC3 best describes deck height relative to constriction potential and bridge type.

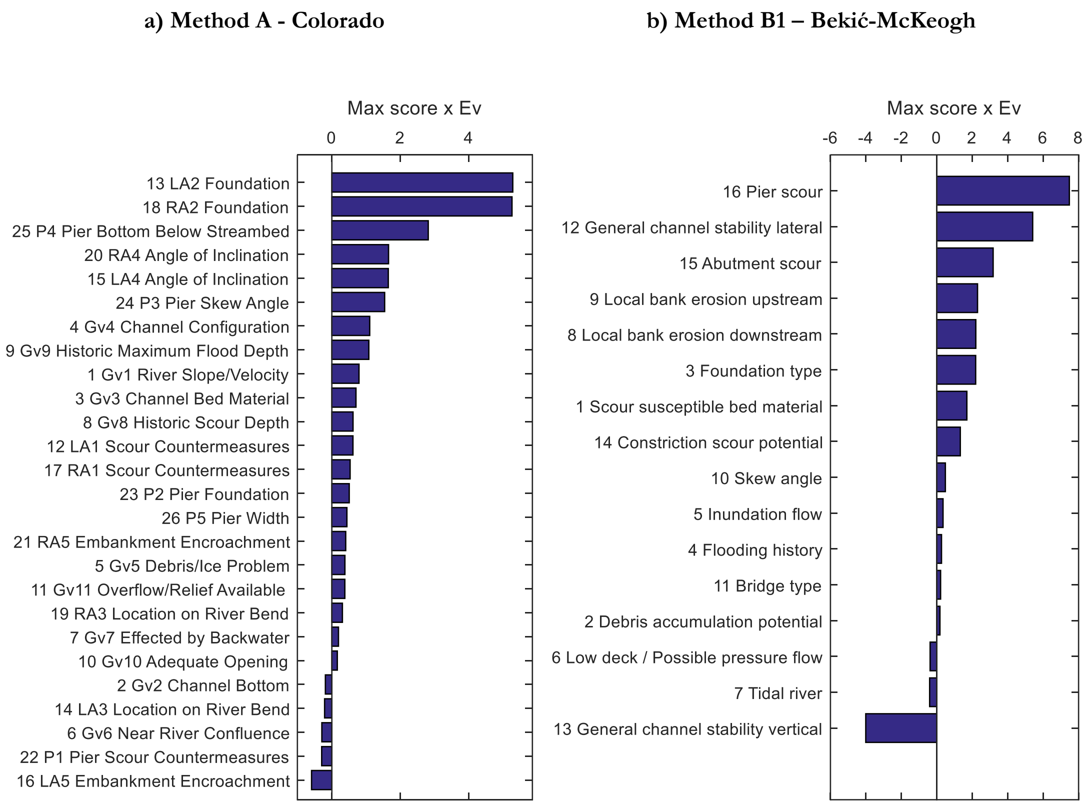


Figure 5.4. Variable maximum scores multiplied by eigenvalues (Ev).

Table 5.3. Eigenvalues, percentage variance explained, factor loadings, means and standard errors of the variables for Method A (Colorado).

Princ. Comp.	Eigenvalue (Ev)	% variance explained	Loading	ID-Code-Variable	Mean	Std. error
PC1	7.891	31.86%	0.193	7-Gv7-Effected by Backwater	-0.002	0.008
			0.160	10-Gv10-Adequate Opening	0.034	0.009
			0.379	11-Gv11-Overflow/Relief Available	0.036	0.016
			5.280	13-LA2-LA Foundation	0.577	0.167
			-0.212	14-LA3-LA Location on River Bend	0.021	0.013
			0.533	17-RA1-RA Scour Countermeasures	-0.033	0.030
			5.257	18-RA2-RA Foundation	-0.002	0.171
			0.308	19-RA3-RA Location on River Bend	0.087	0.018
PC2	3.414	13.78%	0.792	1-Gv1-River Slope/Velocity	-0.022	0.032
			0.618	8-Gv8-Historic Scour Depth	0.034	0.036
			1.079	9-Gv9-Historic Maximum Flood Depth	0.036	0.055
			0.616	12-LA1-LA Scour Countermeasures	0.166	0.033
			1.647	15-LA4-LA Angle of Inclination	0.577	0.066
			1.654	20-RA4-RA Angle of Inclination	0.021	0.058
			1.541	24-P3-Pier Skew Angle	0.049	0.069
PC3	3.212	12.97%	-0.189	2-Gv2-Channel Bottom	-0.061	0.012
			-0.292	6-Gv6-Near River Confluence	0.148	0.012
			-0.593	16-LA5-LA Embankment Encroachment	0.441	0.029
			-0.297	22-P1-Pier Scour Countermeasures	0.041	0.019
			0.507	23-P2-Pier Foundation	0.134	0.017
			2.813	25-P4-Pier Bottom Below Streambed	-0.044	0.095
PC4	1.739	7.02%	1.105	4-Gv4-Channel Configuration	0.009	0.040
			0.381	5-Gv5-Debris/Ice Problem	0.089	0.017
PC5	1.406	5.68%	0.404	21-RA5-RA Embankment Encroachment	0.044	0.029
PC6	1.165	4.70%	0.704	3-Gv3-Channel Bed Material	0.489	0.037
PC7	0.902	3.64%	0.439	26-P5-Pier Width	0.149	0.022

Table 5.4. Eigenvalues, percentage variance explained, factor loadings, means and standard errors of the variables for Method B1 (Bekić-McKeogh).

Princ. Comp.	Eigenvalue (Ev)	% variance explained	Factor loading	ID-Code-Variable	Mean	Std. error
PC1	11.29	34.88%	1.698	1-v1-Scour susceptible bed material	0.337	0.068
			2.195	3-v4-Foundation type	0.452	0.082
			0.353	5-v6-Inundation flow	0.105	0.012
			2.203	8-v9-Local bank erosion downstream	0.079	0.087
			2.299	9-v10-Local bank erosion upstream	0.180	0.086
			0.485	10-v11-Skew angle	0.068	0.017
			5.412	12-x22b-General channel stability lateral	-0.123	0.224
			1.328	14-x23-Constriction scour potential	0.361	0.043
			3.180	15-x24a-Abutment scour	0.312	0.086
			7.480	16-x24b-Pier scour	1.181	0.262
PC2	6.84	21.13%	-4.008	13-x22c-General channel stability vertical	0.245	0.195
PC3	3.74	11.57%	-0.378	6-v7-Low deck / Possible pressure flow	0.038	0.021
			0.216	11-v12-Bridge type	-0.002	0.009
PC4	3.28	10.14%	0.267	4-v5-Flooding History	0.072	0.011
PC5	1.95	6.03%	-0.404	7-v8-Tidal river	-0.050	0.018
PC8	0.95	2.93%	0.179	2-v2-Debris accumulation potential	0.069	0.010

The 26 variables/elements from Method A and 16 variables/elements from Method B1 are represented by vectors on a bi-plot of the principal component space (Figure 5.5-Figure 5.7). The bi-plot displays vectors of the correlation coefficients between the bridge elements and the PCs, as well as a scatter plot representing the level of correlation between two PCs for each bridge structure. It can be seen that for Method B1, variables are all positively correlated for PC 1 (Figure 5.5b). This is not the case for Method A, where from more significant variables (elements), abutment foundations are positively correlated in PC1, while variables (elements) describing skew angle are negatively correlated. This implies that if abutment skew angle rises, abutment scour would decrease. This is contrary to scour development physics at the bridge, implying that the coefficients for Method A need to be re-checked. The correlation coefficients with the highest absolute values indicate how much the PC represents certain variables, and the sign describes the relationship between the PC to the actual state. PC 2 shows that all variables are positively correlated in Method A. For Method B1, PC 2 shows that Pier and abutment scour are positively correlated, while general scour (lateral and vertical) are negatively correlated. This implies that if there is no local scour at the bridge, there is a probability that the bridge experiences general scour. This is possible. However it would be wrong to assume that if there is evident local scour at the bridge there is no risk of general scour, as one might conclude from the graph (Figure 5.5b - Figure 5.7b). This exercise confirms how PCA could be useful tool to show the data which would be lost due multi-dimensionality and high data scattering. Although useful, some results should not be accepted as definite and expert interpretation is required.

a) Method A - Colorado **b) Method B1 – Bekić-McKeogh**

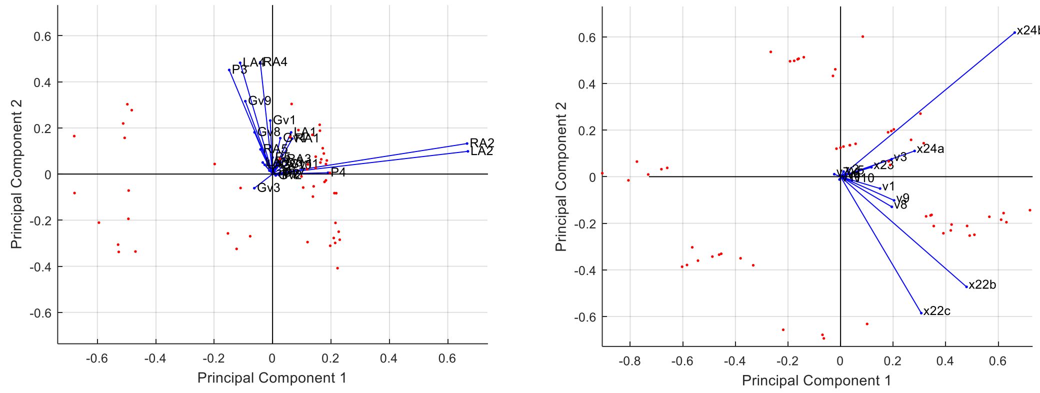


Figure 5.5. Correlation coefficients for PC1 and PC2 for Method A and B1.

a) Method A - Colorado **b) Method B1 – Bekić-McKeogh**

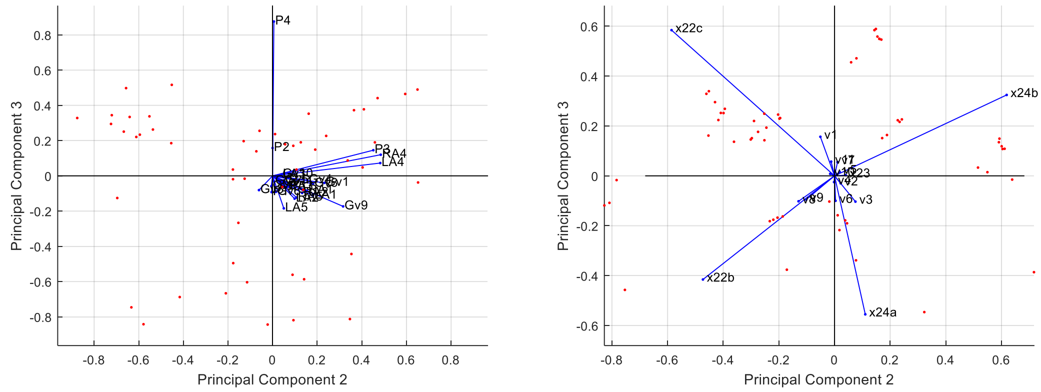


Figure 5.6. Correlation coefficients for PC2 and PC3 for Method A and B1.

a) Method A - Colorado **b) Method B1 – Bekić-McKeogh**

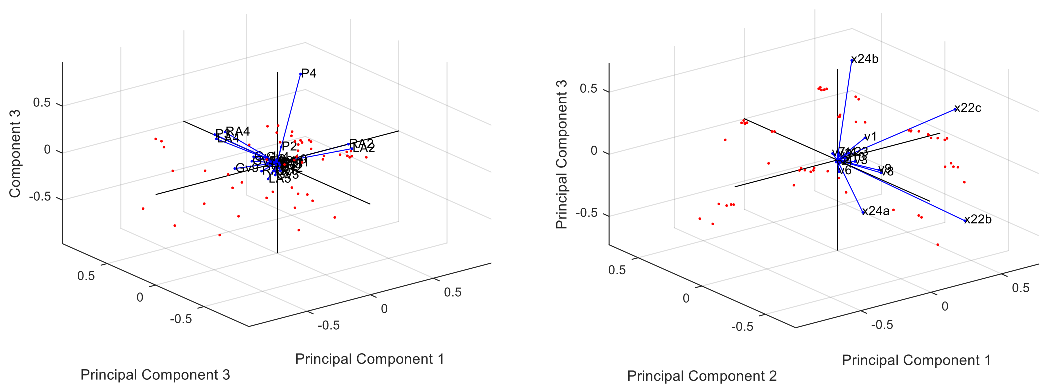


Figure 5.7. Correlation coefficients for PC1, PC2 and PC3 for Method A and B1.

5.3.3 Comparison of Rank Summary with the new scoring system for Method B1

One good example for the comparison between one fast approach such as a scoring system and more detailed approach with expert judgment is bridge UB31 over the river Painestown on the Dublin/Cork line, shown as a highlighted circle (black) in Figure 5.8. When analysing the plot in Figure 5.8 one could consider this bridge as an outlier due to relatively high residual. The bridge is ranked with 40 points (VRS) when using Method A. The bridge was assessed with Rank Summary 30 (Low risk) in Method B1 (without scoring system). However, with a new Scoring system of Method B1, the bridge would have a high total score of 48. This bridge was examined in more detail and it can be seen that in Method B1 a decision was left to the inspector to decide if the bridge would either be ranked as "Low Risk" with the next inspection in 2 years, or as "Stage 2 – Move to Analysis". In the Method B1 with a new scoring system the Bridge indeed has all indications of all three types of scour, but they did not threaten the bridge stability at the time of the inspection. The inspector made a personal judgment to go with lower risk, but with more frequent inspection of the bridge (within two years' time). As both scoring systems (Method A and B1) assign one of the highest ranks, when compared to total score and the other 99 bridges and the approach without scoring system shows medium risk (Low risk with next inspection interval within 2 years) the question arises which of the approaches is more appropriate (with or without scoring system). Surely more detailed analysis such which was applied in Method B1 (without scoring system) is expected to gain more data in order to avoid a highly conservative decision. In the case of UB31, scoring systems would tend to show higher scores (more conservative conclusion) and when taken into account that this method could be significantly faster and more transparent, it should not be abandoned.

By comparing the coefficient of determination ($R^2 = 0.29$) for the regression between VRS and Rank Summary (Figure 5.2) with coefficient of determination ($R^2 = 0.38$) for the regression between Rank Summary and Total score from the new scoring system (Figure 5.8), it can be seen that the correlation was somewhat improved, but the level of improvement is still not satisfactory.

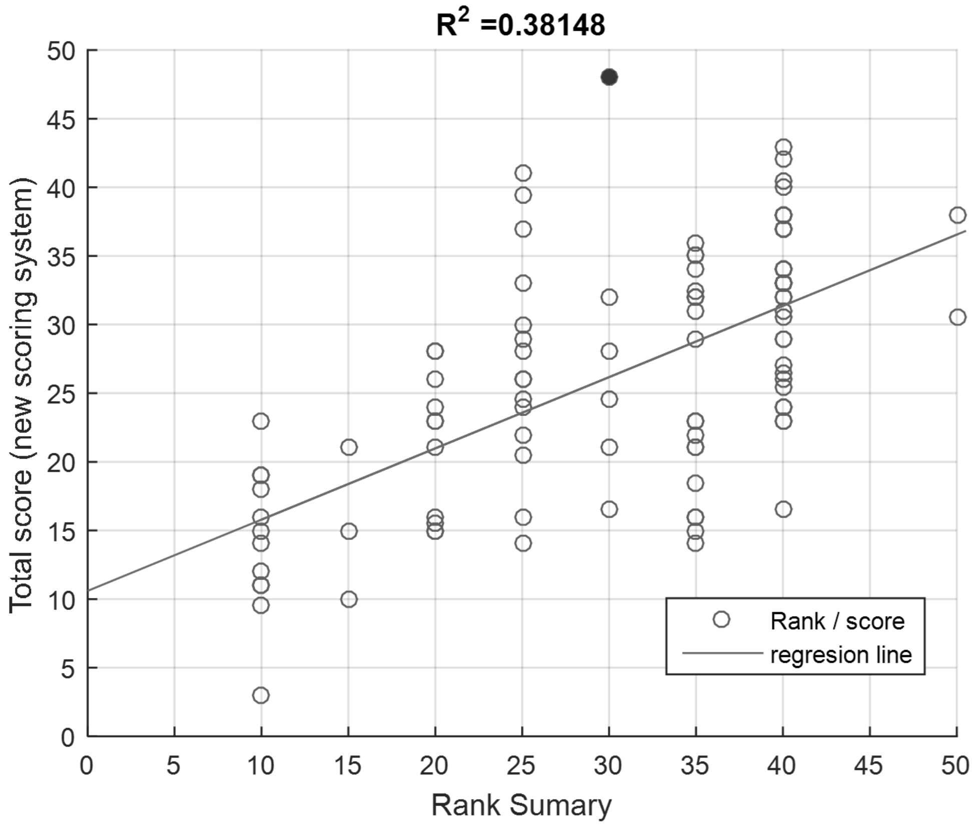


Figure 5.8. Method B1 - Comparison of Rank Summary (RS) with the new scoring system for 100 railway bridges in Ireland.

5.4 Conclusions on PCA

The Principal Component Analysis (PCA) was applied for the

(1) assessment of two bridge scour inspection methods: Method A - Colorado method (in further text Method A) and Method B1 - Bekić-McKeogh Stage 1, referred as Method B1.
(2) Development of a scoring system for the Method B1

PCA could be useful tool to show the data which would be lost due multi-dimensionality and a high data scattering. The majority of the variance is described by first three components Figure 5.3.

Based on the bi-plot graph (Figure 5.5) some discrepancy with scour development physics in Method A is noted. Method A (Figure 5.5a) could imply that if abutment skew angle rises, abutment scour would decrease. As this is contradictory to the physical basis of scour development at the bridge, a conclusion can be made that the coefficients for Method A need to be re-evaluated.

PCA allows to examine if the methods are fundamentally looking at similar aspects when assessing them (by comparing what comprises the principal component, which means are the principal components similarly defined - using bunching of variables). Some differences between Methods can be noted here as well. While Method A gives the highest significance to a Foundation type (Figure 5.4), the significance of historic scour depth is much lower. This is lower even than angle of inclination and skew angle. The most significant elements (variables) Method B1 are local scour, general scour and foundation type. All other elements (variables) have lower significance than evidence on scour and foundation type. From these observations Method A has showed again some inconsistency with the scour problem approach. Method B1 has showed strong appropriateness of the scour evaluation approach, where the comparison of the scour depth (elevation) and foundation depth (elevation) define scour risk.

Comparison of Method A and B1 showed very weak correlation [162] of $R^2 = 0.29$ (see Figure 5.2). Findings in [126, 142] and PCA analysis above confirms that Method B1 overpowers the Method A.

In order to overcome the main restrictions in Method B1 (the need for standardisation of scoring system and the fact that the report writing process requires significant amount of time and resources), PCA was utilised in order to develop the new scoring system for Method B1.

The PCA showed to be a very useful tool when defining the scoring system for the scour assessment system. The iterative process enabled to determine the final number of elements (variables) in the scoring system (some elements needed to be excluded from the assessment as they were showing higher significance than expected). PCA also enabled to determine the scores of each element (variable). The new scoring system for Method B1 is shown in Table 5.2.

As an overall conclusion, PCA was successfully utilised for the comparison of the two scour assessment methods (Method A and B1) and was proven to be useful tool to develop a new scoring system for Method B1 (Table 5.2). However, the newly developed scoring system for Method B1 has low correlation ($R^2 = 0.38$) with the old scoring system and consequently cannot be recommended for further use.

In order to standardise the scoring system, reduce the subjectivity of inspectors, incorporation of expert experience, lowering the time of reporting and resources needed to complete the inspection, it is recommended to fundamentally change the Method B1 assessment or create a completely new scour assessment approach, e.g. a new scour inspection module.

This conclusion introduces Chapter 6, supported by Annex J and Annex K, which outlines the new inspection Method, e.g. inspection module.

Chapter 6
Development of Inspection Module

6.1 General description

As indicated in Chapter 4 and Chapter 5, in order to standardise the scoring system, eliminate the subjectivity of inspectors, bring in experts' experience, and lower the time of reporting and resources needed to complete the inspection it is recommended to make radical changes to the existing approach to scour assessment and inspection. As a result, the development of a new bridge scour inspection module will be described in detail in this Chapter. The methodology and approach described below is a result of more than three years work under the umbrella of the EU FP7 Project Bridge SMS (www.bridgesms.eu) and it is applicable for bridges over waterways. All bridges also need to be assessed to determine their structural condition. Structural inspection of bridges is not a subject of this thesis as it is well documented and is in application (see sections 3.1 and 3.2.2)

The inspection module is part of the overall Bridge Management system. The module is integrated within the Bridge Management System web-based platform and it is equipped with a mobile (tablet) application currently available for Android Operating System.

The proposed module is an enhancement and addition to any existing structural bridge inspections, specifically designed for bridges over water with unknown or shallow foundations.

The proposed Scour Condition ratings are shown in Table 6.1. The overall Condition Rating (CR) should be the highest of the Structural Condition Rating (StCR) and Scour Condition Rating (ScCR).

The Scour Condition Ratings (ScCR) are augmented with the recommended year to next inspection (see Table 6.6 and Table 6.7).

Table 6.1 Scour Condition Ratings

Scour Condition Rating ScCR	Description
ScCR:0	No or insignificant damage.
ScCR:1	Minor damage but no need of repair.
ScCR:2	Some damage, repair needed when convenient. Observe the development of the condition.
ScCR:3	Significant damage, repair or scour risk management needed very soon, i.e. within next financial year.
ScCR:4	Damage is critical. It is necessary to execute repair works or scour risk management at once.
ScCR:5	Ultimate damage. The component has failed or is in danger of total failure, possibly affecting the safety of traffic. It is necessary to implement Plan of Action (PoA) without delay after the introduction of limitation measures.

6.1.1 Assessment process

Assessment of a structure should be carried out in levels of increasing complexity, with the objective to efficiently determine its adequacy. Level 0 Appraisal comprises a check-up of a structure during routine patrol on roads/railways. Level 1 General Inspection comprises simple methods, including the use of engineering judgement, to identify structures that have no major structural and scour defects or where the defects are tolerably low.

- Provided that a structure is shown to be adequate for Level 1 Inspection then the assessment is complete.
- Where a structure is not adequate for Level 1 Inspection or shows major structural and scour defects, then the assessment should progress to Level 2.
- When a structure needs additional information or management of structural and scour risks, then the assessment will progress to Level 3.

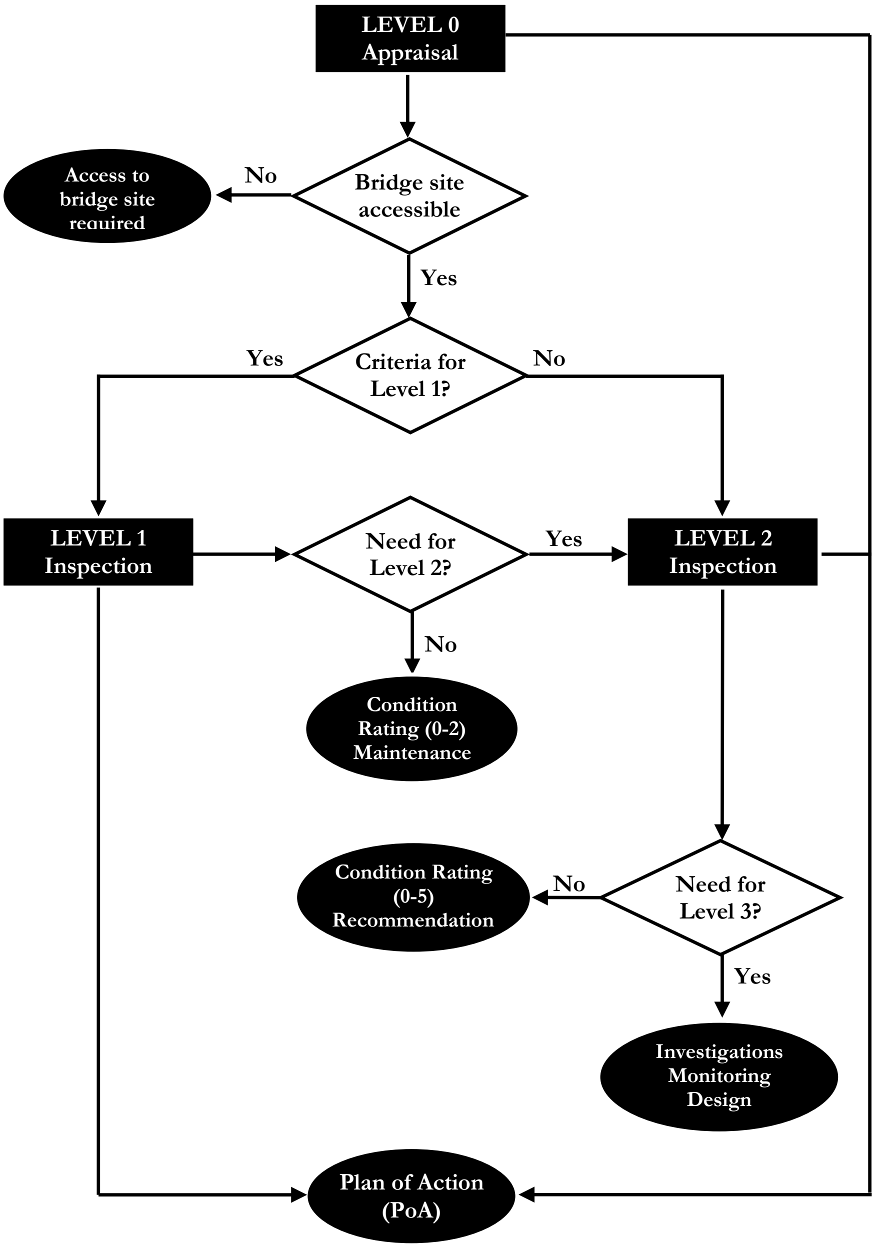


Figure 6.1 Assessment process

6.1.2 Types of inspections

BRIDGE INVENTORY

Before any of the inspections can proceed, the bridge inventory needs to be developed. This step can be conducted by office and field screening of the bridges. For older bridges, that are not previously listed in any database, a field screening is required. This level of bridge screening should collect limited number of data such as: road/railway, bridge geographical location, basic geometry elements (number of spans and piers), bridge over the river, etc. For that purpose, a mobile (tablet) application is designed as part of the proposed procedure.

LEVEL 0 – ROUTINE APPRAISAL

Level 0 Routine Appraisal is carried out by trained technician on a routine check-ups of roads/railways. It includes short notes in a simple form. The output from the appraisal may include the followings:

- Recommendations for gaining access to the bridge site.
- Alert on failure of structural and/or scour components (Plan of Action, PoA).
- Recommendations for routine maintenance (e.g. removal of debris).

LEVEL 1 – GENERAL INSPECTION

Level 1 General Inspection comprises general structural and general scour inspections for Level 1 bridges and is carried out by a trained inspector (trained area engineer). It is a standardised inspection and report performed on site with no need for an office desk study. The assessment comprises of visual and tactile inspection of the structure and river channel, state assignment for structural and scour components, notes on damages and photo documentation production. It is carried out in time intervals of 2 up to 6 years. The recommended inspection team comprises of a trained inspector and a technician. The outputs from general inspection include the following:

- Recommendations for gaining access to the bridge site.
- Alert on failure of structural and/or scour components (PoA).
- Structural Condition Rating: StCR:0 to StCR:2 or recommendation to proceed to Level 2 inspection.
- Scour Condition Rating: ScCR:0 to ScCR:2 or recommendation to proceed to Level 2 inspection.

- Recommendations for maintenance works (i.e. repair of components).
- Time for next inspection.

Level 1 General inspection is fully implemented within the mobile (tablet) application.

LEVEL 2 – DETAILED INSPECTION

The Level 2 Detailed Inspection is carried out for Level 2 briges by a certified engineer. The assessment consists of detailed structural and scour inspections and each inspection requires certificate(s) (structural inspection certificate and scour inspection certificate). Besides the site visit, the inspection requires an office desk study. The assessment comprises visual and tactile inspection of structure and river channel, data gathering from other sources, state assignment for structural and scour components, notes on damages and photo documentation production. It is carried out in time intervals of 2 to 6 years.

Level 2 Detailed Inspection is fully implemented within the mobile (tablet) application.

LEVEL 3 – INVESTIGATION, MONITORING AND DESIGN

Level 3 Investigation, Monitoring and Design is carried out by field experts for selected bridges from the Level 2 inspection. It may include collection of more detailed information (desk study, investigation, monitoring, modelling, design) such as: foundation investigation, underwater inspection, bed material investigation, velocity surveys, soil layer profiling, material testing, load capacity assessment, non-destructive testing, scour monitoring, water level monitoring, hydraulic modelling, structural modelling, slope stability assessment and modelling, scour protection design, structural design and geotechnical design.

Table 6.2 Types of inspections

LEVEL OF INSPECTION	STAFF REQUIREMENTS (BRIDGE INSPECTOR)	INSPECTION INTERVAL	USUAL OUTPUTS
BRIDGE INVENTORY	Trained technician	Only once	• road/railway, • bridge geographical location, • basic geometry elements (number of spans and piers), • is the bridge over the river/waterway
LEVEL 0 - ROUTINE APPRAISAL	Trained technician	No requirements	• Recommendation for gaining access to the bridge site. • Alert on failure of structural and/or scour components (PoA). • Recommendations for routine maintenance.
LEVEL 1 – GENERAL INSPECTION	Trained inspector	From 1 up to 6 years	• Recommendation for gaining access to the bridge site. • Alert on failure of structural and/or scour components (PoA). • Structural Condition Rating: StCR:0 to StCR:2 or proceed to Level 2. • Scour Condition Rating: ScCR:0 to ScCR:2 or proceed to Level 2. • Recommendations for maintenance works. • Time for next inspection.
LEVEL 2 – DETAILED INSPECTION	Certified structural inspector Certified scour inspector	From 1 up to 6 years	• Recommendation for gaining access to the river. • Alert on failure of structural and/or scour components (PoA). • Structural Condition Rating: StCR:0 to StCR:5 • Scour Condition Rating: ScCR:0 to ScCR:5 • Recommendations for further investigations
LEVEL 3 – INVESTIGATION, MONITORING AND DESIGN	Structural expert Hydraulic expert Geotechnical expert	As required by Level 2 Inspection	• Material testing, foundation investigations, etc. • Structural monitoring, scour monitoring, etc. • structural design, hydraulic modelling,

6.1.3 Coding of components and elements

The bridge scour condition rating (ScCR) and structural condition rating (StCR) are obtained by inspecting all bridge components. Some bridge components comprise of several elements. Assessment of components and elements is performed by assigning the "state" of a component/element.

Table 6.3 State of bridge components and elements

A - GOOD	B - FAIR	C - POOR	D - SEVERE	E - CRITICAL	F - FAILED	N/A

Table 6.4 Coding of bridge components and elements

Level of inspection	Type of inspection	Component code	Element code
L1 (Level 1)	L1.Sc (Level 1 Scour insp.)	c01-c99	e01-e99
L2 (Level 2)	L2.Sc (Level 2 Scour insp.)	c01-c99	e01-e99

6.1.4 Naming and numbering convention

The naming and numbering of the bridge components is relative to the direction of the flowing water when looking downstream (in the direction of the flow). If the bridge is located on the tidal section of the river, an ebb tide (Fluvial River) direction is the relevant direction of the flow for the naming convention. It is possible that during the site visit the direction of the flow is difficult to be determined or it is possible that the person on site will misjudge, and potentially to get an impression that the flow is in the opposite direction to the normal direction of the flow (small streams, in the vicinity of lakes, tidal conditions, etc.). In these cases information should be retrieved from maps (historical maps, topographical maps, orthophoto maps, etc).

Following the downstream orientation during inspection the naming and numbering convention:

- river channel, bridge elevation, floodplain = upstream / downstream
- abutment = left / right
- river bank, wing wall = upstream left / upstream right / downstream left / downstream right
- span, pier = from the left to the right hand side, starting with Span 1 / Pier 1 and with increasing numbers to the right hand side

6.1.4.1 Unification with other naming systems

The proposed naming convention detailed aboves may differ from already accepted naming convention used in other countries or institutions which would mainly rely on the ordinal directions, direction of the road relative to town/city (direction to Dublin, direction to Cork, etc.), mileage of the road/railway, etc. Structural inspections would mainly use the ordinal direction naming convention (N, E, S, W, NE, SE, SW, NW), see Figure 6.2. In order to unite two different naming conventions, two mandatory fields are introduced:

- Ordinate Flow Direction

Where the bridge inspector needs to define the direction of the flow of the river (if the river flows from North West to South East following marking would be assigned: NW-SE). See blue arrow in example shown in Figure 6.2.

- Ordinate Road Direction

Where the bridge inspector defines the road direction in accordance with the rule relative to flow direction: the left abutment is named first (example: N) and right abutment is named second (S). The example input would then look like this: N-S. See red arrow in example shown in Figure 6.2. When bridge is over the stream/river, numbering of piers should be always from left to right relative to the flow.

Figure 6.2 Compass rose for identification of the orientation of the flow and road (with example).

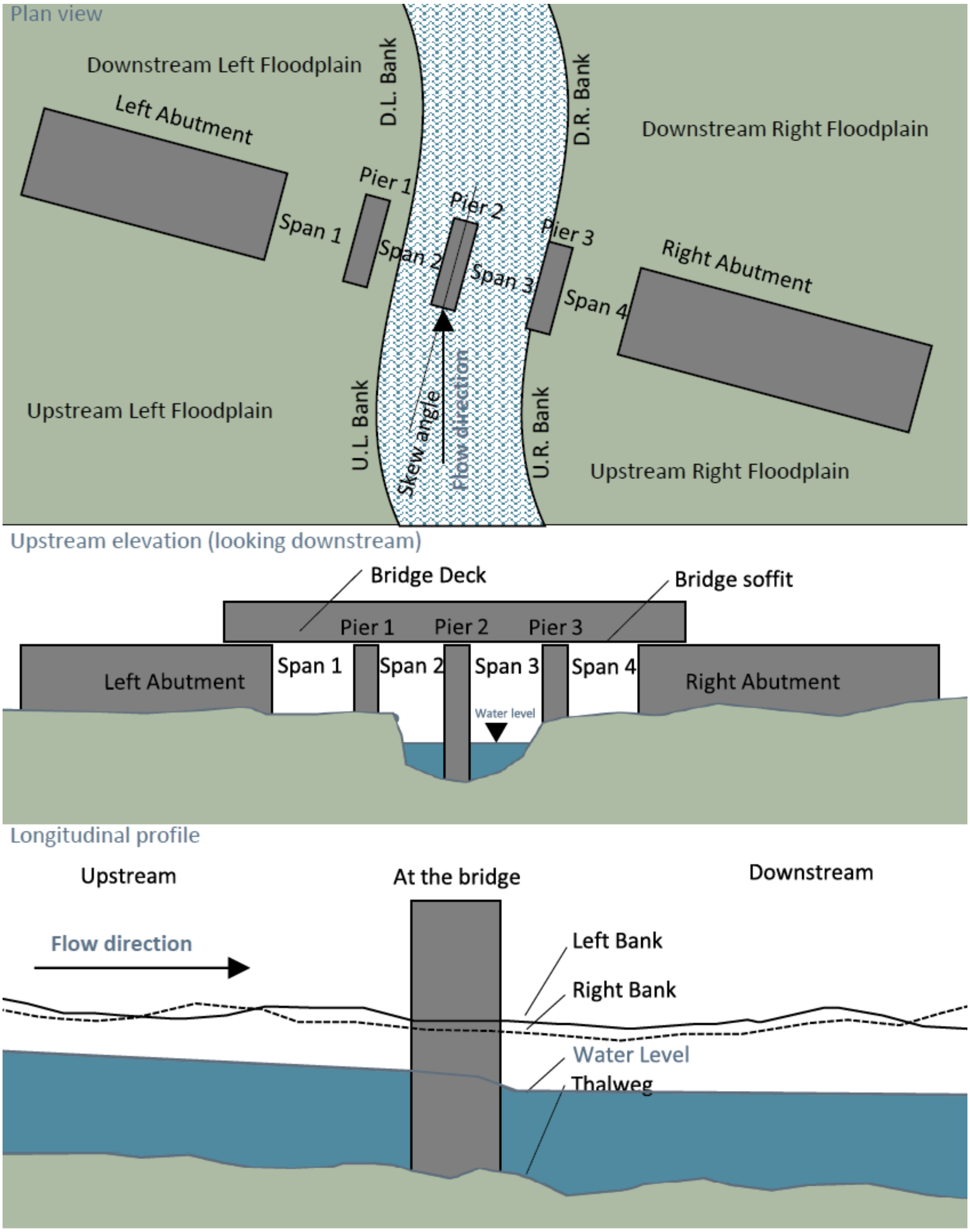


Figure 6.3 Naming and numbering convention of bridge elements

6.1.5 Inspection route and photo documentation

The inspection route and photo documentation is different for Scour and Structural inspections.

6.1.5.1 Scour inspection route and photo documentation

Photo documentation is an important part of any bridge inspection. Generally, it is recommended to record as few as possible photos per bridge inspection, but with more features in the photo (zoom-out). This normally includes photo of upstream and downstream bridge profile, and upstream and downstream river channel respectively. Additional photos should be obtained of any noted damage to scour components, such as channel scour, bank scour, scour protection damage, etc. It is also recommended to obtain a photograph of the riverbed material with a scale (measuring tape, etc.) depending on site conditions.

Figure 6.4 Example of photos from the bridge scour inspection

The following procedure and order[7] of the bridge inspection photographic documentation is recommended:

1. A photo of the structure identification (Photo 0) needs to be recorded, usually located at the bridge road wall/railing, which has been placed directly on the structure (in order to be able to recognise the bridge in question).
2. When standing at the bridge deck[8] (footpath or road) the following pictures should be obtained:
    - Photo 1: Upstream river channel, looking upstream
    - Photo 2: Downstream river channel, looking downstream
    - If evident, any notable defects (rails, surfacing, drainage, etc.) should be also documented
3. Approach the bridge from the upstream side either from the upstream left or upstream right bank, depending on the site conditions (vegetation, more suitable angle to see the bridge, etc.). Use following order for photo documentation
    - Photo U1: Upstream elevation of the bridge
    - Photo U2: Left abutment, wing-wall and upstream left bank
    - Photo U3.1: Pier 1 (if applicable)
    - Photo U3.2: Pier 2 (if applicable)
    - Photo U3.n: Pier "n" (if applicable)
    - Photo U4: Right abutment, wing-wall and upstream right bank
    - Photo U5: River bed material (upstream)
    - Photo U6: Other defects (cracks, debris, etc.)
    - Photo U7: Other Structures (channels, weirs, embankments, etc.)
4. When the inspector obtains all photographs from the upstream side, he goes to the downstream side of the bridge. Similar to the upstream side, following order of photographs is obtained with the difference that the inspector now faces upstream:
    - Photo D1: Downstream elevation of the bridge
    - Photo D2: Left abutment, wing-wall and downstream left bank (looking upstream)
    - Photo D3.1: Pier 1 (if applicable)
    - Photo D3.2: Pier 2 (if applicable)
    - Photo D3.n Pier "n" (if applicable)
    - Photo D4 Right abutment, wing-wall and downstream right bank (looking upstream)
    - Photo D5 River bed material (upstream)
    - Photo D6 Other defects (cracks, debris, etc.)
    - Photo D7 Other Structures (channels, weirs, embankments, houses, etc.)

The suggested inspection route is shown in Figure 6.5.

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[7] The described order of photographic documentation is recommended, however other routes can be followed
depending on accessibility in the bridge site and the site conditions at the time of the visit.

[8] This can be done for road and pedestrian bridges, as usually walking on the or near the railway line would be forbidden. For railway bridges, when walking near the line is not possible as there are no footpaths, photographs should be obtained from upstream/downstream side of the bridge, facing in the opposite direction of the bridge elevation.

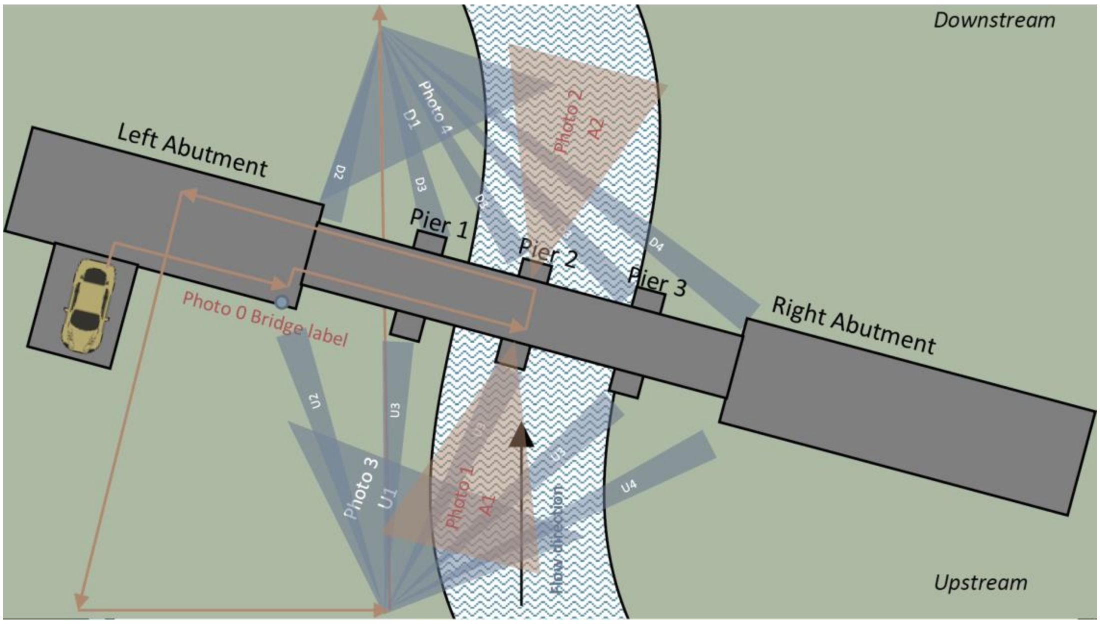


Figure 6.5 The suggested route for bridge scour inspection.

6.1.6 Survey Reference Point, Survey Grid and Time to next inspection

6.1.6.1 Sketches for typical permanent reference point for riverbed elevation

a. Description

When riverbed bathymetry is measured (i.e. cross-section, scour depth, overbank elevation) or elevation of specific bridge elements (i.e. foundation depth, soffit clearance, pile cap elevation) the measurement must refer to common elevation datum in order to obtain absolute elevations. When the engineer measures elevation of some of the aforementioned components during inspection, a simple measuring tape or levelling rod will be used rather than advanced geodetic equipment. These measurements will be collected using relative height frame referenced to a clearly visible and easily distinguishable point or feature on the bridge structure (i.e. bridge deck, piers or abutments).

In order to determine absolute elevations, the engineer will have to rely on a known permanent reference point to convert relative measurements into absolute elevations. Permanent reference points may differ from site to site, depending on the bridge type, bridge clearance height and water depth or pier shape. Assignment of suitable permanent reference point is up to the engineer for each site, to place it on a visible location that can be easily recognized and approached with geodetic equipment in order to obtain its absolute elevation if needed. Once the engineer assigns the appropriate location for permanent point it must be marked by coloured spray paint and photographed for future reference. Below are listed typical points that can be selected for several bridge types, depending on their geometry.

b. Sketches of typical reference points

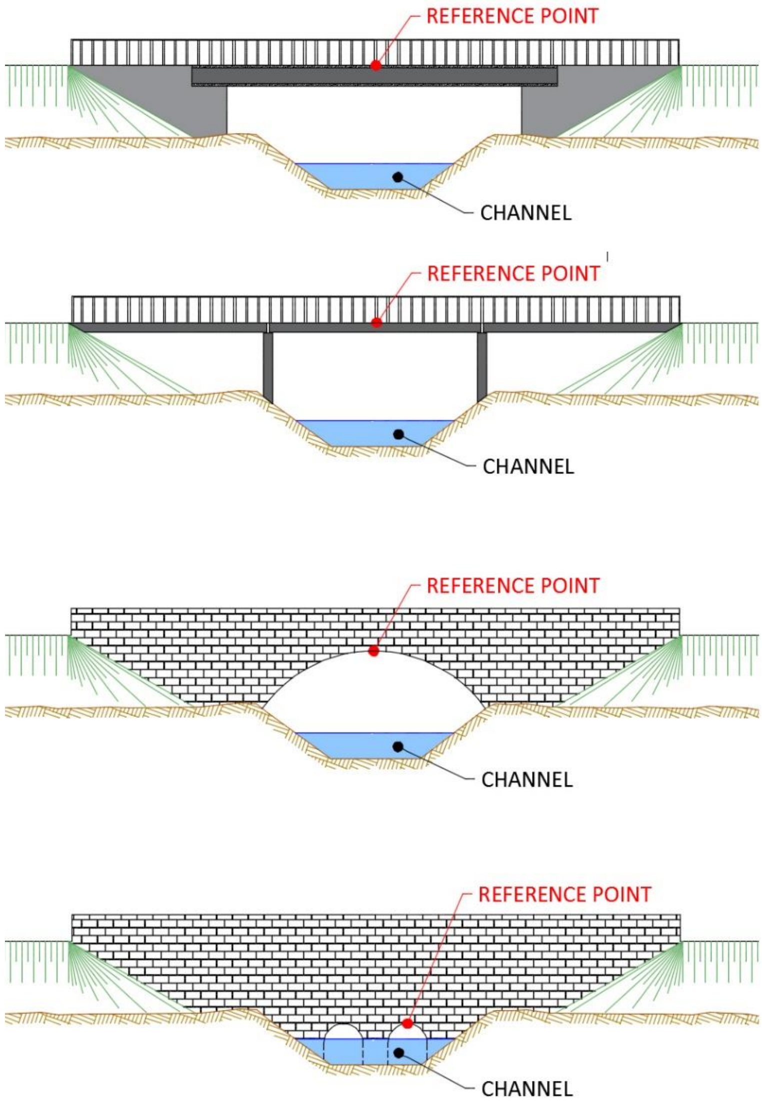


Figure 6.6 Sketches of typical reference points based on bridge type.

6.1.6.2 Survey Grid

As part of inspection a small survey needs to be obtained. In a case when the river bed is accessible, survey can be performed without a boat. The results of survey can be presented in relative coordinate system (relative to the reference point, see chapter 6.1.6.1). For instance, the height values of points are relative to 100m.

If the river bed survey requires a boat, then all points should be georeferenced and presented in the absolute coordinate system. For instance, the height values are presented relative to ordinance datum (mOD).

For the relative coordinate system, a survey grid with a coding system is developed (see Figure 6.8). The survey grid shows minimum desirable number of points per bridge. The survey grid can be used to take all the points shown on the grid, just few points or the points can be even more detailed. When a point is measured during the bridge inspection it is mandatory to refer to the location of the point in accordance with the Survey Grid (Figure 6.8). The coding system is explained below (Figure 6.7). The number of parallel lines (3 lines) in Figure 6.8 is valid if the bridge span is less than 5m. The minimum number of lines relative to span length is shown in Table 6.5.

Describes number of span
1 - Span 1
2 - Span 2
…

Describes number of line (ID) parallel to the flow (from left to right):
1 - line 1
2 - line 2
3 - line 3
…

Describes number of the line (ID) perpendicular to the flow (from upstream to downstream)
1 - Point 1
2 - Point 2
…
7 – point 7

1 - 3 - 7

Figure 6.7 Coding system for the Survey grid.

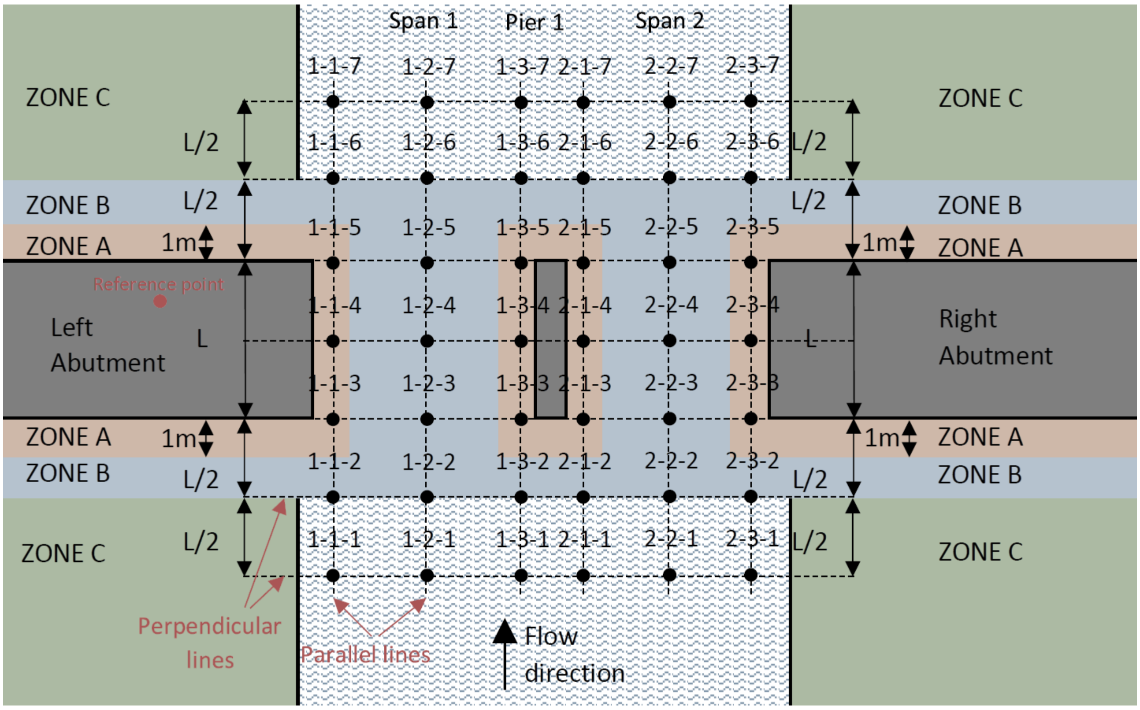


Figure 6.8 Survey grid.

Table 6.5 Number of survey lines (parallel) relative to span length

Span length [m]	Number of survey lines parallel to the flow
<5	3
5-10	5
10-15	7
15-20	9
>20	Detailed survey

6.1.6.3 Recommended time to next inspection

The recommended values for the years to next inspection are linked to a Level 1 and Level 2 ScCR and are described in tables below This recommendation is calculated automatically, as indicated in the tables below. However, the procedure allows the bridge inspector to provide an additional safety factor to the assigned scour condition rating ScCR. The recommendation for the years to next inspection is obtained by bridge inspector based on the "in-house" policies of the bridge management / organisation.

For bridges with unknown foundations, it is recommended to do inspections every 2-3 years. For specific cases it is also possible to recommend longer time to next bridge inspection. The bare minimum to inspect the bridge is from 6years.

Table 6.6 Time for next inspection for Level 1 bridges.

<table>
<tr><th>Scour Condition Rating (ScCR)</th><th>Bridge with known foundations</th><th>Bridge with unknown foundations</th></tr>
<tr><td>0</td><td>6</td><td>6</td></tr>
<tr><td>1</td><td>6</td><td>6</td></tr>
<tr><td>2</td><td>6</td><td>4</td></tr>
<tr><td>3</td><td colspan="2" rowspan="3">Level 2</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
</table>

Table 6.7 Time for next inspection for Level 2 bridges.

<table>
<tr><th rowspan="2">Scour Condition Rating (ScCR)</th><th colspan="2">Bridge with known foundations</th><th colspan="2">Bridge with unknown foundations</th></tr>
<tr><th>Survey with boat</th><th>Survey without boat</th><th>Survey with boat</th><th>Survey without boat</th></tr>
<tr><td>0</td><td>6</td><td>6</td><td>4</td><td>6</td></tr>
<tr><td>1</td><td>6</td><td>6</td><td>4</td><td>6</td></tr>
<tr><td>2</td><td>6</td><td>6</td><td>4</td><td>4</td></tr>
<tr><td>3</td><td>4</td><td>4</td><td>4</td><td>4</td></tr>
<tr><td>4</td><td colspan="4" rowspan="2">At least every 1 year until completion of works</td></tr>
<tr><td>5</td></tr>
</table>

6.6 Inspection module and mobile App

The above described methodologies were implemented within the tablet application (in further text referred to as the mobile App) for bridge inspections.

The mobile App brings the inspection to a completely different level as it enables a full use of Information and Communication Technology (ICT) for reducing the time of the inspection and more efficient transfer and management of the information. The bridge inspections are not just reports stored in a folder or in someone's drawer, but they are directly connected to the database and the overall BMS platform. In this way the history of the inspections and condition of the bridge can be backtracked. By backtracking of the condition of the bridge possible inspector subjectivity could be accommodated as every inspection needs to be approved by a supervisor, and changes are possible even after inspection. After the inspection is approved, the data collected from the field becomes active within the BMS platform (database). The information of the inspection is used as part of Decision Support System. The BMS platform enables the creation of .pdf reports from the Inspection.

One more important element of the application is photo documentation. The photo documentation is standardised (in the form of a checklist). Photographs with descriptions are automatically assigned to a bridge and river elements. Historically photographs could be misplaced (e.g. photographs stored in an incorrect location), or due to corporate memory loss, meaning some of the photographs could be lost. As an example, even a couple of days after completion of the inspection, the inspector would not always be sure in which direction the photo is facing (upstream or downstream), o the erosion of the bank shown on the photo is on the left or right bank, etc.

The App works in an offline mode and results from the inspections can be synchronised whenever an internet connection is established. The mobile application access is restricted and is available only on request.

6.7 Conclusions on Inspection module

The newly developed scour assessment methodologies was developed during and under supervision of the BRIDGE SMS Project (PI Dr. Eamon McKeogh and Project Co-PI Dr. Damir Bekić).

The Author of this Thesis had a major input into the method described in this chapter with constant communication and help of producing component description and some sketches for component states shown as annex to this thesis (Annex J and Annex K).

The inspection module for both Level 1 and Level 2 was tested on-site with help from Project partners Cork Co.Co. and Infrastructuraes de Portugal and external engineers from Malachy Walsh and Partners, see Annex D.

This thesis had a major input in a development of a mobile application for the Level 1 and Level 2 bridge inspections. The mobile application was developed by Project partner ARCTIS d.o.o. The Author of this Thesis was continuously supporting development of the mobile application and online web-platform in period between 2016-2020.

The method is already applied on limited number of bridges in Ireland, Portugal and Croatia. However a more extensive and detailed application is required. Thus, the following chapter (Chapter 7) applies proposed procedure for L1 and L2 scour inspections on 100 bridges railway bridges in Ireland. Results of evaluation conducted in Chapter 7 have very high significance for acceptance or further improvement or rejection of methodology demonstrated in Chapter 6.

Chapter 7
Evaluation of Inspection Module

The implications and importance of the evaluation of a new Inspection Module, e.g. the bridge scour inspection methodology were mentioned in the conclusions of the previous chapter (Chapter 6). After the development of a new method, this is the key assessment that will provide an answer to whether the proposed method(s) are applicable or need further improvement. It should be noted that the math for calculating of Scour Condition Rating (ScCR) provided in Excel spreadsheets:

- Level 1: L1.ScCR: "Annex Ja - L1_Scour Inspection Form.xlsx"
- Level 2: L2.ScCR: "Annex Ka - L2_ScourMatrix_v05b-Template.xlsm",

enables transparent adjustment of weighting factors for calculation of L1 ScCR and lookup matrices for calculation of L2 ScCR.

The results of the following section are gained based on a series of adjustments and sensitivity analysis of the weighting factors (for L1) and lookup matrices (for L2) firstly through the collaboration with researchers on the BRIDGE SMS project and then as part of the iterative process for the sample of 101 railway bridge in Ireland.

Although this thesis will provide tested and verified weight factor(s) and lookup matrices, further adjustment is recommended if a dataset with even larger sample(s) of bridges becomes available.

The most relevant results of verification of methods L1 and L2 are shown in this chapter. Significant additional work and pair-wise comparisons are outlined in Annex L and Annex M.

7.1 Method for evaluation

The two proposed levels of inspection – Level 1 (L1) and Level 2 (L2) were compared with a method B1 in this chapter (Chapter 7). A detailed pair-wise comparison of methods L1 and L2 with four existing methods Method B1, B2a, B2b and C (see Chapter 4) is shown in Annex L. Schematics of pair-wise comparison is shown in (Table 7.1). The comparison was done using parametric and non-parametric regression analysis. Parametric regression analysis was conducted by plotting the method results on a scatter plot and calculation of Pearson's coefficient "r" and the coefficient of determination (R^2), e.g. square of the Pearson's correlation coefficient. The size of the dots on scatter plots represent the number / percentage of the bridges with the same ratings. A non-parametric regression analysis was conducted by calculating Spearman's correlation coefficient, the Kendall Tau-b Correlation Coefficient, and the Hoeffding Dependence Coefficient. Detailed correlation analysis with corresponding "p" values for each regression coefficient is shown in Annex M.

Thresholds to determine the goodness of fit, defined by Moore [162] for the coefficient of determination will also be applied for absolute values of Pearson's, Spearman, Kendall correlation and Hoeffding correlation:

- $R^2 < 0.3$ very weak fit,
- $0.3 < R^2 < 0.5$ weak fit,
- $0.5 < R^2 < 0.7$ moderate fit,
- $R^2 > 0.7$ strong fit.

Of four methods for comparison, Method B1 is the crucial method for comparison. The rationale for selecting the B1 method for comparison is listed below:

- representative sample for the selected method
- previous inspections carried out by at least two senior engineers
- the previous bridge inspections are considered to be reliable.

Thus, a hypothesis is defined that "the method X[9] is plausible and recommended for application on bridges if the coefficient of determination R^2 is close to 1.0 when comparing the method X with Method B1".

[9] By method X it is meant any of the methods (Method L1, L2, C, B2a or B2b) that will be compared to method B1

Tests of the number and percentage of acceptable and not acceptable outcomes (see Table 11.52, Table 11.53 and Table 11.54) of comparisons between methods is shown in Annex L. In case that the percentage of non-acceptable outcomes is larger than 5%, these methods will not be considered to be significantly dependent.

7.2 Theoretical background - correlation coefficients

7.2.1 Pearson Product-Moment Correlation Coefficient "r"

The Pearson product-moment correlation is a parametric measure of association for two variables. It measures both the strength and the direction of a linear relationship. If one variable y is an exact linear function of another independent variable x, a positive relationship exists if the correlation is 1 and a negative relationship exists if the correlation is –1. If there is no linear predictability between the two variables, the correlation is 0. If the two variables are normal with a correlation 0, the two variables are independent. However, correlation does not imply causality because, in some cases, an underlying causal relationship might not exist.

The equation for the sample Pearson product-moment correlation is (eqn 7.1):

$$r_{xy} = \frac{\sum_i [(x_i - \bar{x})(y_i - \bar{y})]}{\sqrt{\sum_i (x_i - \bar{x})^2 \sum_i (y_i - \bar{y})^2}} \qquad \text{(eqn 7.1)}$$

Where $\bar{x}$ is the sample mean of x, and $\bar{y}$ is the sample mean of y.

The Pearson's coefficient value is in range from -1 to +1, with zero value meaning there is no correlation.

Probability values of Pearson correlation are computed by equation (eqn 7.2) below:

$$p = (n-2)^{0.5} \left(\frac{r^2}{r - r^2} \right)^{0.5} \qquad \text{(eqn 7.2)}$$

Where t is probability from t-distribution with (n-2) degrees of freedom, where r is the sample Spearman Correlation.

7.2.2 Coefficient of determination "R²"

The coefficient of determination R^2 tells us what proportion of variation in *y* is explained by variation in *x*. The general expression of coefficient of determination (eqn 7.3) is:

$$R^2 = 1 - \frac{SE_{res}}{SE_{tot}} \quad \text{(eqn 7.3)}$$

Where SE_{res} is the squared error (residuals) with respect to the linear regression line and SE_{tot} squared error (residuals) with respect to the average value of *y* ordinate of dependant variable.

In a simple linear regression model case, the coefficient of determination equals the square of the value of the Pearson correlation coefficient "r". That is valid in the case where we treat x_i and y_i as a random sample from a joint distribution, e.g. $R^2 = r^2$. The coefficient of determination value is in the range from 0 to +1, with zero value meaning there is no correlation.

7.2.3 Spearman Rank-Order Correlation

The Spearman coefficient [163] represents rank-order correlation as a nonparametric measure of association based on the ranks of the data values. The calculation of the coefficient is shown in equation (eqn 7.4) below:

$$\theta = \frac{\sum_i[(R_i - \bar{R})(S_i - \bar{S})]}{\sqrt{\sum_i(R_i - \bar{R})^2 \sum_i(S_i - \bar{S})^2}} \quad \text{(eqn 7.4)}$$

Where, R_i is a rank of x_i, S_i is a rank of y_i, $\bar{R}$ is mean of R_i values and, $\bar{S}$ is the mean of S_i values.

The value of the Spearman coefficient value is in the range from -1 to +1, with zero value meaning there is no correlation.

The probability values of the Spearman correlation are computed by the equation (eqn 7.5) below:

$$p = (n-2)^{0.5}\left(\frac{\theta^2}{\theta - \theta^2}\right)^{0.5} \quad \text{(eqn 7.5)}$$

Where *t* is the probability from the t-distribution with (n-2) degrees of freedom, where θ is the sample Spearman Correlation.

7.2.4 Kendall's Tau-b Correlation Coefficient

Kendall's Tau-b coefficient [163] is a nonparametric measure that is based on the number of concordances (when paired observations vary together) and discordances (paired observations vary differently) in paired observations. The coefficient is calculated using equation (eqn 7.6) below:

$$r_{\tau-b} = \frac{\sum_{i<j}\left[sgn\left(x_i - x_j\right)sgn\left(y_i - y_j\right)\right]}{\sqrt{(T_0 - T_1)(T_0 - T_2)}} \qquad \text{(eqn 7.6)}$$

Where, $T_0 = n(n-1)/2$, $T_1 = \sum_k t_k(t_k - 1)/2$, and $T_2 = \sum_i u_i(u_i - 1)$; t_k is the number of tied x values in the kth group of tied x values, u_i is the number of tied y values in the ith group of tied y values, *n* is the number of observations, and sgn(z) is defined as sgn(z) = 1 (if z>0) or sgn(z) = 0 (if z=0) or sgn(z) = -1 (if z<0).

The value of the Kendall coefficient lie in the range from -1 to +1, with a zero value meaning there is no correlation.

The probability values for Kendall's tau-b are computed by the equation (eqn 7.7) below:

$$p = \frac{s}{\sqrt{V(s)}} \qquad \text{(eqn 7.7)}$$

Where $s = \sum_{i<j}\left[sgn\left(x_i - x_j\right)sgn\left(y_i - y_j\right)\right]$, and *V(s)* is the variance of *s*.

7.2.5 Hoeffding Dependence Coefficient

Hoeffding's measure of dependence, *D* [163], is a nonparametric measure of association that detects more general departures from independence. The statistic approximates a weighted sum over observations of chi-square statistics for two-by-two classification tables [164]. Each set of values are cut points for the classification. The equation (eqn 7.8) for Hoeffding's D is:

$$D = \frac{(n-2)(n-3)D_1 + D_2 - 2(n-2)D_3}{n(n-1)(n-2)(n-3)(n-4)} \qquad \text{(eqn 7.8)}$$

$D_1 = \sum_i (Q_i - 1)(Q_i - 2)$,
$D_2 = \sum_i (R_i - 1)(R_i - 2)(S_i - 1)(S_i - 2)$, and
$D_3 = \sum_i (R_i - 2)(R_i - 2)(S_i - 2)(Q_i - 1)$.

Where, R_i is rank of x_i, S_i is rank of y_i, and Qi (also called bivariate rank) is 1 plus the number of points with both x and y values less than i^{th} point.

The Hoeffding coefficient value is in the range from -0.5 to +1, with +1 value indicating there is a complete dependence between the variables.

The probability values for Hoeffding's D statistic are computed using the asymptotic distribution computed by Blum, Kiefer, and Rosenblatt [165], see equation (eqn 7.9) below:

$$p = \frac{(n-1)\pi^4}{60} D + \frac{\pi^4}{72} \qquad \text{(eqn 7.9)}$$

Where n is the sample size and D is Hoeffding's coefficient.

7.3 Data identification

The method of the evaluation is conducted for three data blocks (DB1-DB3). The difference between the blocks is in the type of the bridges – Data block 1 includes only simple (single) span bridges, while Data block 2 includes both, single and multi-span bridges. Data block 3 is introduced as for the 25 railway bridges (part of data block 2), methods B2a and B2b were applied. Method C was applied on the same 101 railway bridges as Method B1. This was possible as the input data required for Method C was extracted from 101 reports from bridge scour inspections for which Method B1 was applied. Method C is firstly compared with Method B1. After this step, legibility of the Method C for verification of methods L1 and L2 will be assessed. In Table 7.1 a comparison of the schemes is graphically shown. The Table shows the matrix for comparison of the methods, also the numbers "1,2,3" refers to the data block for which comparison was applied. This chapter will outline only comparison between methods B1, L1 and L2. A detailed pair-wise comparison is shown in Annex L. The input data for three data blocks with calculation of correlation coefficients is shown in Annex M. A more detailed description of each data block (DB) is shown in the following section.

Table 7.1. Comparison matrix showing the comparing scheme between the methods.

Method	B1	L1	L2	C	B2a	B2b
B1	n/a	DB1	DB1, DB2	DB1, DB2	DB3	DB3
L1	DB1	n/a	DB1	DB1	DB3	DB3
L2	DB1, DB2	DB1	n/a	DB2	DB3	DB3
C	DB1, DB2	DB1	DB2	n/a	DB3	DB3
B2a	DB3	DB3	DB3	DB3	n/a	DB3
B2b	DB3	DB3	DB3	DB3	DB3	n/a

*DB1, DB2 and DB3 represent the data block for which comparison was made

7.3.1 Data block 1 – 44 single span bridge

A sample of 44 bridges single span bridges in Ireland from the UCC study commissioned by Irish Rail was used [126, 142]. The bridges were previously prioritised using UCC-Bekić-McKeogh Stage 1 method. As stated above, the methodology is applied on forty-four (44) single span railway bridges in Ireland (see Figure 7.2). The span length for analysed 44 bridges ranges between 1.0m and 38.5m (11.1m average), see Figure 7.1. The river width ranges between 1.0m and 35.0m (10.4m on average). The table with the details on Bridges are in Annex J.

Table 7.2. Summary statistics showing mean value, standard deviation, mean and maximum value and size (N) of the sample for Data block 1.

Method (grade)	Mean	Std Dev	Minimum	Maximum	N
Method B1 (PR)	2.09091	0.56314	1	3	44
Method L1 (ScCR)	1.79545	0.90424	0	3	44
Method L2 (ScCR)	1.88636	0.92046	0	4	44
Method C (Category)	3.97727	0.69846	3	5	44

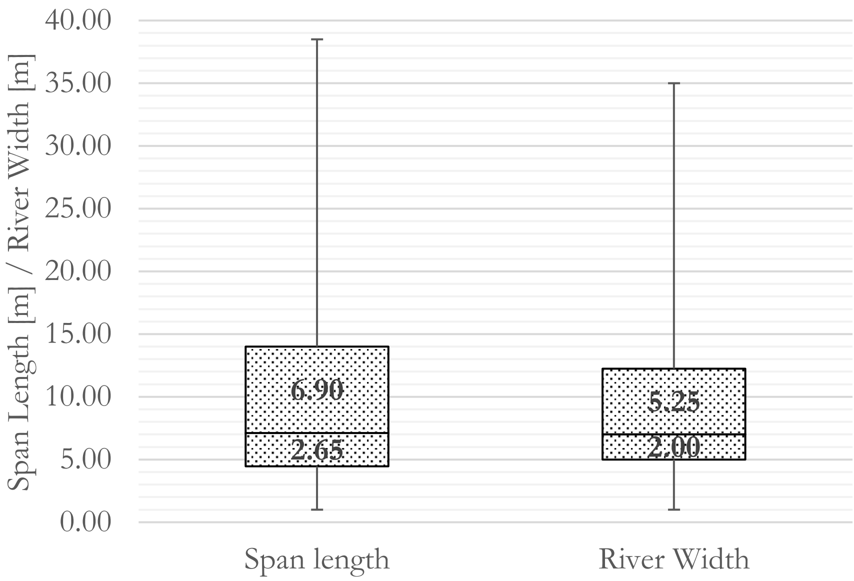


Figure 7.1. Boxplot showing Bridge Span lengths and River Lengths for Data block 1.

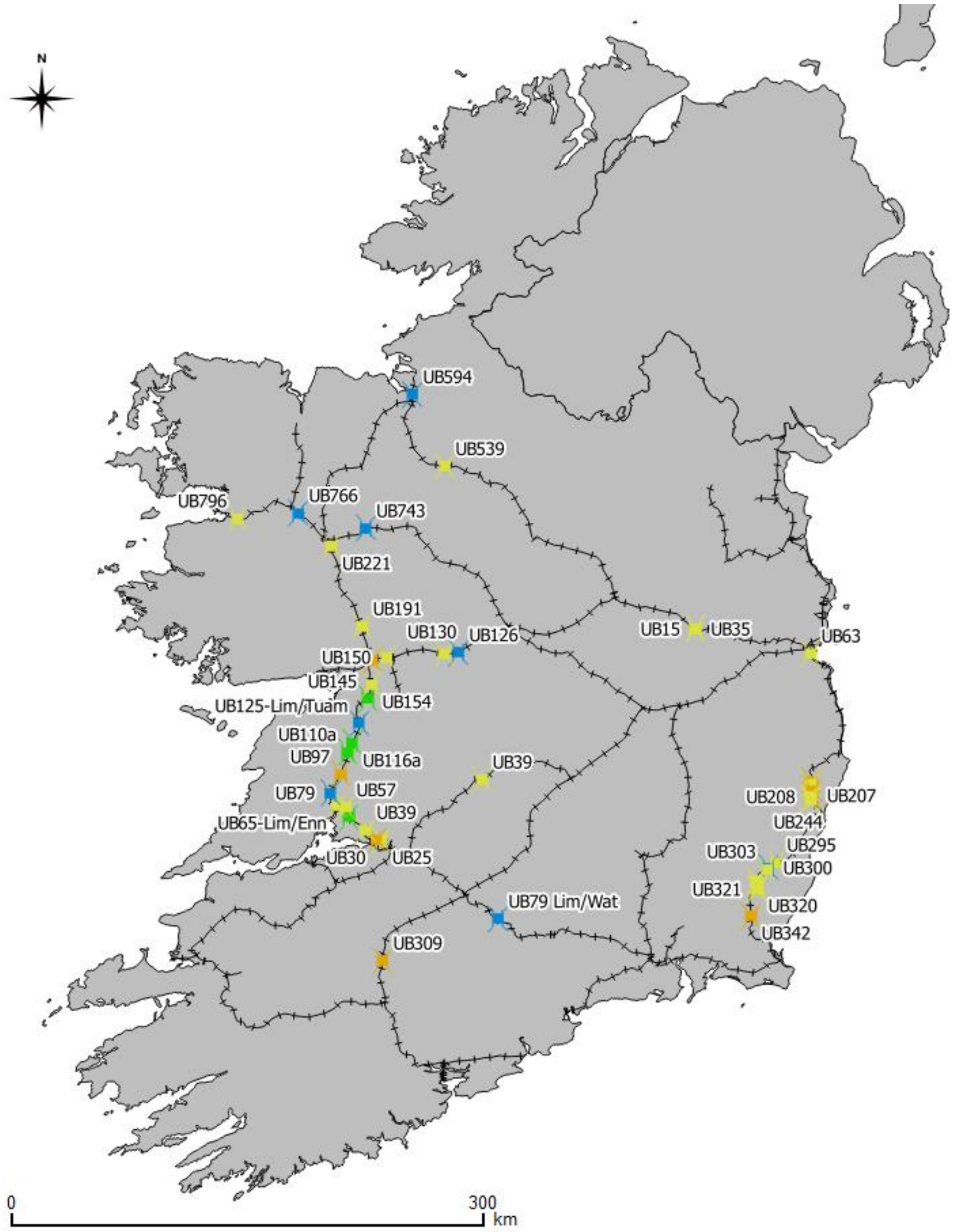


Figure 7.2. Data block 1 - Location of bridges (44).

7.3.2 Data block 2 – 101 single and multi-span bridge

A sample of 101 single and multi-span bridges in Ireland from the UCC study commissioned by Irish Rail was used [126, 142]. The bridges were previously prioritised using the UCC-Bekić-McKeogh Stage 1 method.

Table 7.3. Summary statistics showing mean value, standard deviation, mean and maximum value and size (N) of the sample for Data block 2.

Method (grade)	Mean	Std Dev	Minimum	Maximum	N
Method B1 (PR)	2.25743	0.65808	1	4	101
Method L1 (ScCR)	1.79545	0.90424	0	3	44
Method L2 (ScCR)	2.11881	1.00287	0	5	101
Method C (Category)	4.00990	0.67075	3	6	101

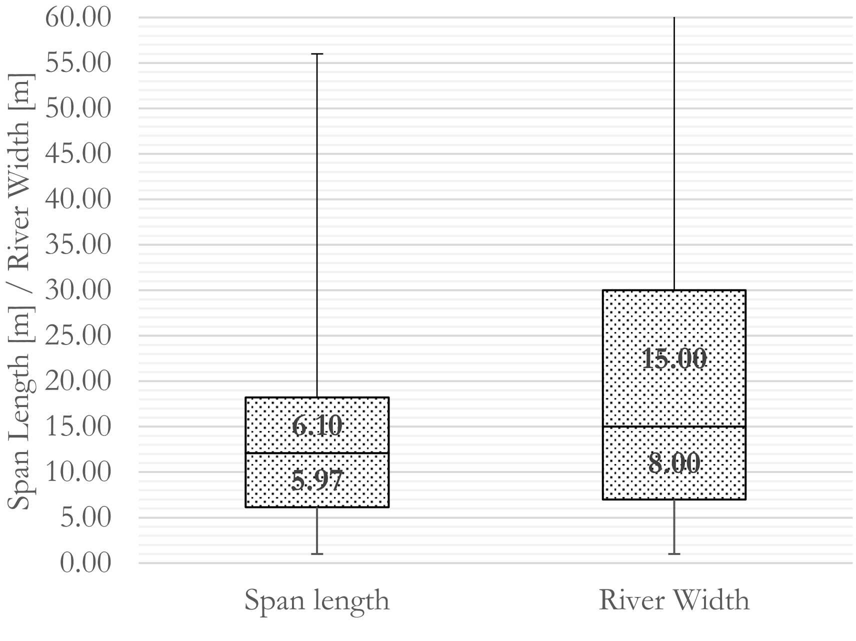


Figure 7.3. Boxplot showing Bridge Span lengths and River Lengths of input bridges for Data block 2.

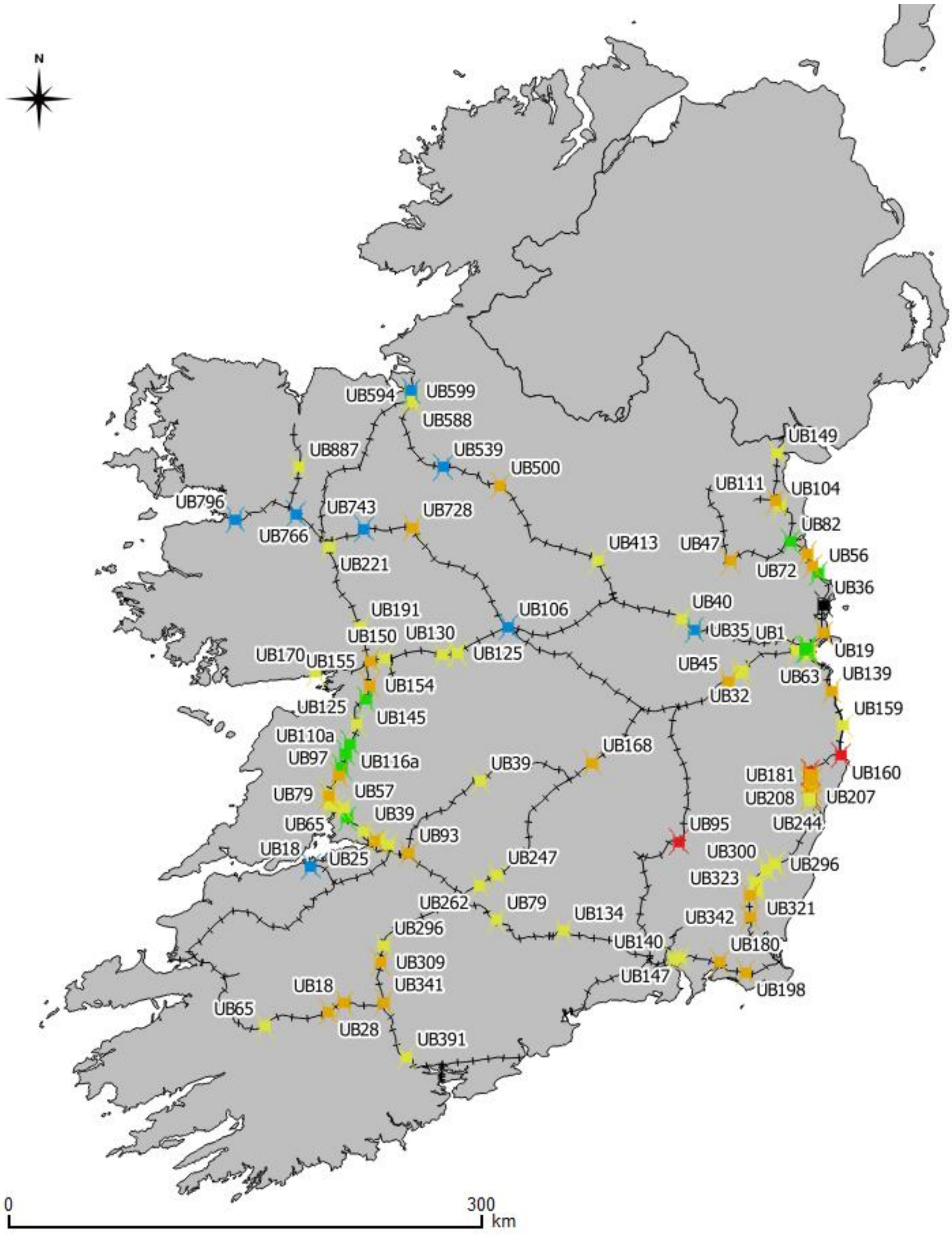


Figure 7.4. Data block 2 - Location of bridges (101).

7.3.3 Data block 3 – 25 stage 2 bridges

Block 3, consisting of 25 railway bridges in Ireland was introduced as on this dataset methods B2a and B2a were applied. The dataset was created for Malachy Walsh and Partners and Fluvio R&D study commissioned by Irish Rail was used.

Table 7.4. Summary statistics showing mean value, standard deviation, mean and maximum value and size (N) of the sample for Data block 3 .

Method (grade)	Mean	Std Dev	Minimum	Maximum	N
Method B1 (PR)	3.00000	0.00000	3	3	25
Method L1 (ScCR)	3.04000	0.45461	2	4	25
Method L2 (ScCR)	3.72000	0.61373	3	5	25
Method C (Category)	2.36000	0.99499	1	5	25
Method B2a (SRR)	12.16000	3.88029	6	21	25
Method B2b (QR)	3.00000	0.00000	3	3	25

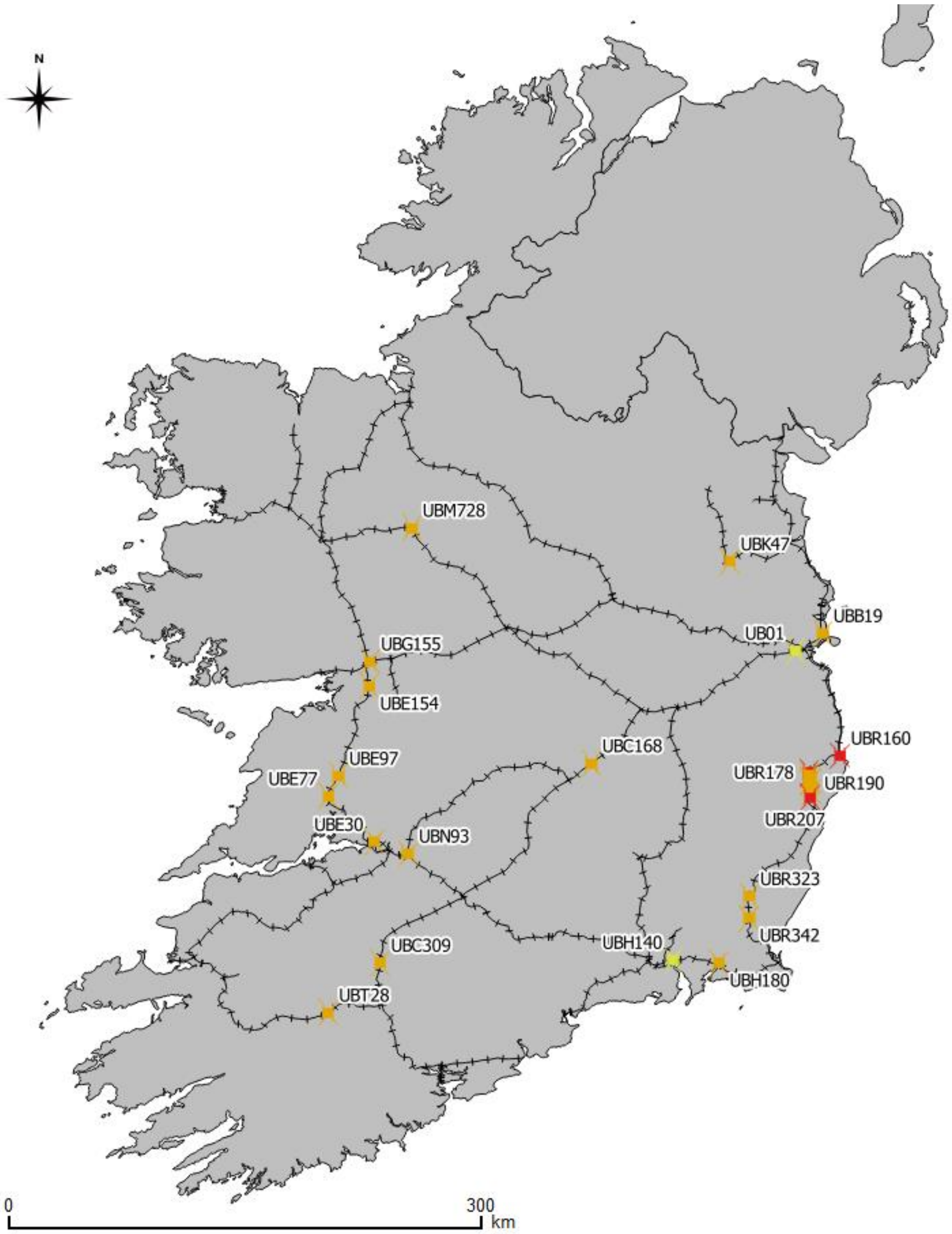


Figure 7.5. Data block 3 - Location of bridges (25).

7.4 Summary of results and Conclusions

This section will outline the results of comparison of methods L1 and L2 with method B1. A more detailed pair-wise comparison between methods B1, L1, L2, B2a, B2b and C is shown in Annex L and Annex M.

7.4.1 Comparison results between Method B1 and L1

7.4.1.1 Comparison notes

The comparison is made for 44 single span railway bridges across Ireland. Priority Rating (PR) obtained by Method B1 (Modified BA74/06 Bekić-McKeogh) is compared to Method L1 Scour Condition Rating (L1.ScCR) Table 7.5 below shows preferable (expected) outcomes when Method B1 and Method L1 are applied on the same bridge.

Table 7.5. DB 1 - Matrix showing when the results of scour inspections are comparable

	Expected results from bridge inspections for the same bridge Priority rating / L1.ScCR and L2.ScCR			
Method B1 **Priority Rating** **(PR)**	1 Insignificant Risk.	2 Low risk (maintenance, minor actions).		3 Move to Stage 2 - Analysis.
Method L1 **Scour Condition Rating (L1.ScCR)**	0 No or insignificant damage.	1 Minor damage but no need of repair.	2 Some damage, repair needed when convenient.	3[10] Proceed to Level 2 inspection.

This means that if the result of the bridge inspection is Priority Rating PR = 1 (in the case when Method B1 is applied), then the expected result for applying Method L1 (Scour Inspection for Level 1 Bridges) should be Level 1 Scour Condition Rating of L1.ScCR = 0 (No or insignificant damage). The Method L1 refined the Priority Rating PR 2 "Low

[10] In Level 1 Bridge Scour Inspection, Scour Condition Rating ScCR 3 is not assigned, yet the bridge is recommended to Proceed to Level 2 Scour Inspection

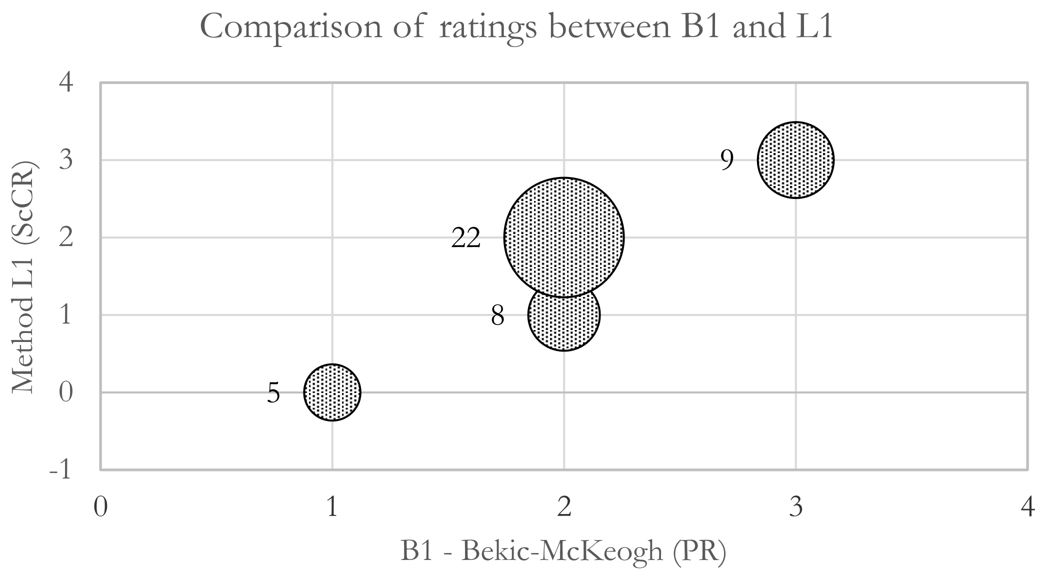
Risk" into two Condition Ratings L1(L2).ScCR 1 "Minor damage but no need of repair" and L1(L2).ScCR 2 "Some damage, repair needed when convenient". This finer refinement of Scour Condition Rating (when compared with Priority Rating PR 2) makes a more appropriate distinction between bridges with potential scour risk (L1.ScCR 1) and bridges where there is an evidence of scour risk (L1.ScCR 2) which could be mitigated with some minor repair works. Very similar, staged, approach is used for Methods B1 and L1. If bridge has PR3 "Move to the Stage 2 Analysis" assigned using Method B1, it is expected that Method L1 recommends "Proceed to Level 2 inspection (Method L2)".

7.4.1.2 Results

The results (Figure 7.6) indicate that correlation (R^2 = 0.82) between Method B1 and Method L1 (L1.ScCR) is strong [162]. Note that the bubble size in Figure 7.6 presents the number and percentage of the bridges respectively. For five bridges (11.4%) that had PR 1 (insignificant risk) from Method B1, Method L1 assigned Scour Condition Rating L1.ScCR 0 (No or insignificant damage). For bridges that Method B1 gained PR 2 (Low risk), Method L1 assigned Scour Condition Rating L1.ScCR 1 (Minor damage) for eight bridges (18.2%) and L1.ScCR 2 (Some damage) for 22 bridges (50.0%). For nine bridges (20.5%) that had PR 3 (Move to Stage 2 – Analysis), Method L1 recommended to Proceed to Level 2 inspection (L2). No discrepancies from the preferred results are noted (see Table 7.5).

During the inspection procedure it was noted that for one bridge (2.3%), Method L1 could potentially underestimate the bridge Scour Condition Rating. This is noted for the bridge named "UB154" over Craughwell River on the Limerick/Tuam railway line. If the inspector's decision was to opt for state C, the bridge Condition Rating of L1.ScCR = 2 would be lower than Method B1 PR = 3 due to a marginal decision for a component 8 (L1.Sc.c8). The recorded scour depth is between 0.5m and 0.6m, implying the inspector needs to decide between state C ("Scour depth <0.6m or sedimentation present. Bank erosion <1.0m to bridge abutments") and state D (Scour depth >0.6m or undermining of bridge abutments, go to Level 2). The final decision was to opt state D, so in the final results there is no difference between the ratings of Method B1 and Method L1. However, this implies that last four components (L1.Sc.c8, L1.Sc.c9, L1.Sc.c10 and L1.Sc.c11) have more significance to the final results than the previous seven components (L1.Sc.c1 to L1.Sc.c7).

The overall conclusion from the comparison is that zero bridges, e.g. (0%) of the results using Method L1 have unacceptable results.



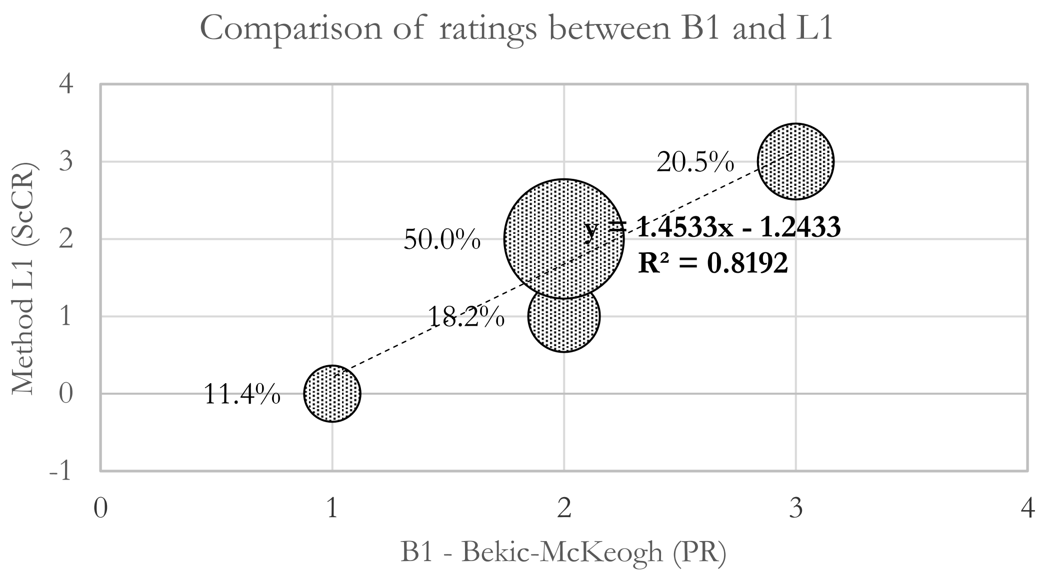


Figure 7.6 Results of the comparison between Method B1 and L1 for Data block 1.

7.4.2 Comparison results between Method B1 and L2

7.4.2.1 Comparison notes

The comparison is made for 101 railway bridges across Ireland. The main purpose of this comparison is to validate Method L2 with Method B1. The comparison is made between Priority Rating (PR) obtained by Method B1 (Modified BA74/06 Bekić-McKeogh) and Method L2 Scour Condition Rating (L2.ScCR) from the newly developed Scour Inspection Module.

Table 7.6 below shows anticipated outcomes when Method B1 and L2 are applied on the same bridge.

Table 7.6. DB 2 - Matrix showing when the results of scour inspections are comparable

<table>
<tr><td></td><td colspan="7">Expected results from bridge inspections for the same bridge Priority rating / L2.ScCR</td></tr>
<tr><td>Method B1 Priority Rating (PR)</td><td>1
Insignificant Risk.</td><td colspan="2">2
Low risk
(maintenance, minor actions).</td><td colspan="2">3
Move to
Stage 2 - Analysis.</td><td colspan="2">4
Immediate action required (PoA).</td></tr>
<tr><td>Method L2 Scour Condition Rating (L2.ScCR)</td><td>0
No or insignificant damage.</td><td>1
Minor damage but no need of repair.</td><td>2
Some damage, repair needed when convenient.</td><td>3
Significant damage, repair needed within next financial year.</td><td colspan="2">4
Damage is critical. It is necessary to execute repair works or scour risk management at once</td><td>5
Ultimate damage. The component has failed or is in danger of total failure</td></tr>
</table>

This means that if the result of the bridge inspection is Priority Rating PR = 1 (in case when Method B1 is applied), then the expected result for applying the Method L2 (Scour Inspection for Level 2 Bridges) should be Scour Condition Rating of L2.ScCR = 0 (No or insignificant damage). The Method L2 has refined the Priority Rating PR 2 "Low Risk" into two Condition Ratings L2.ScCR 1 "Minor damage but no need of repair" and L2.ScCR 2 "Some damage, repair needed when convenient".

For PR 3 (Move to Stage 2 Analysis) from Method B1, acceptable Scour Condition Ratings in Method L2 would be L2.ScCR 3 (Significant damage) or L2.ScCR 4 (Damage is critical). Lastly, for PR 4 (Immediate action required) in Method B1, acceptable Scour Condition Ratings in Method L2 would be L2.ScCR 4 (Damage is critical) or L2.ScCR 5 (Ultimate damage), in accordance with Table 7.6 below.

7.4.2.2 Results

The results (Figure 7.7) indicate that correlation (R^2 = 0.82) between Method B1 and Method L1 (L1.ScCR) is strong [162]. Note that the bubble size in figures below presents the number and percentage of the bridges respectively. For ten bridges (9.9%) that gained Priority Rating PR 1 (Insignificant risk) from Method B1, Method L2 evaluated all of those bridges with Scour Condition Rating L2.ScCR 0 (No or insignificant damage). For fifty-seven bridges (56.4%) that gained Priority Rating 2 (Low risk) from Method B1, Method L2 evaluated eight bridges (7.9%) with L2.ScCR 1 (Minor damage) and forty-seven bridges (46.5%) with L2.ScCR 2 (Some damage).

For two bridges (2.0%) that have PR = 2, Level 2 assigned one level higher Scour Condition Rating L2.ScCR 3 (Significant damage). This will be looked at in more detail in paragraph below.

This case occurred for two bridges with internal Irish Rail reference UB45 and UB500 bridge. UB45 is a 2 span girder bridge on masonry abutments over the River Liffey, west of Sallins in Co. Kildare and UB500 is a 6-span bridge located in the middle part of the River Shannon. Both of the bridges have unknown foundations.

For the UB45 bridge, local scour depth of 0.5m was evident. Method B1 cannot compare scour depth with foundation depth in case foundations are unknown. Method L2 has the ability to compare scour depth even for unknown foundations, for rules see Table 6.11. Decision for priority rating for UB45 using method B1 was marginal, between PR 2 and PR 3. It was concluded solely on expert opinion to go with lower rating of PR 2, but with recommendation that the bride is re-inspected within next 2 years. Method L2 recommended inspection in the next 4 years.

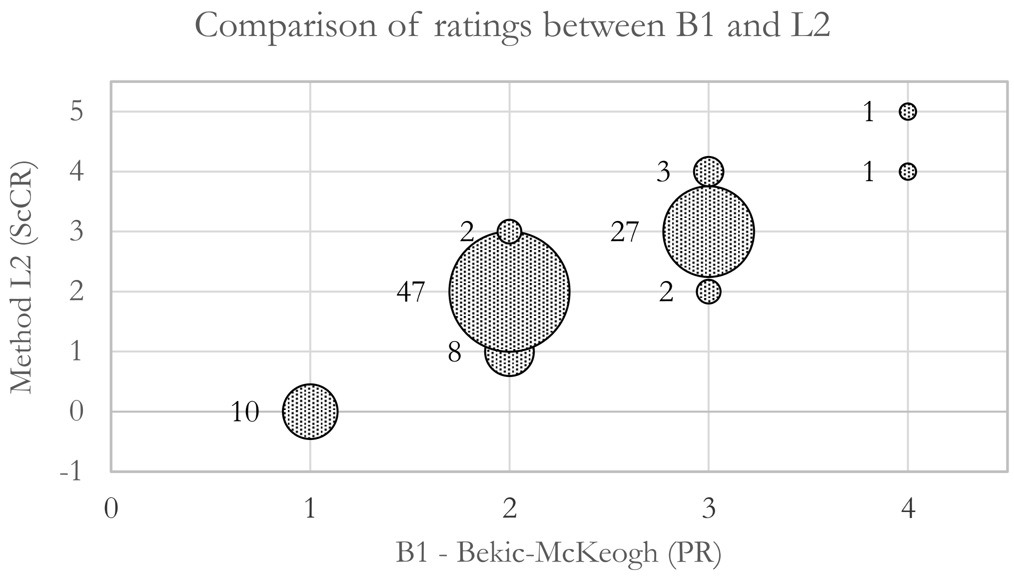
For the UB500 the same, marginal decision was made between PR 3 and PR 2 when method B1 was applied. Expert gave a subjective opinion that the bridge could be assigned with PR = 2, but that re-inspection should be done within one year. Method L2 relied on comparison of scour depth and estimated foundation depth Table 6.11 and an overall Condition Rating of 3 was assigned with recommendation for re-inspection within four years.

After more detailed analysis, this discrepancy is justified as being acceptable.

For thirty-two bridges (31.7%) that gained Priority Rating 3 (Move to Stage 2 Analysis) from Method B1, Method L2 evaluated twenty-seven bridges (26.7%) with L2.ScCR 3 (Significant damage) and three bridges (3.0%) with L2.ScCR 4 (Damage is critical). For two bridges (2.0%) that have PR = 3 (Move to stage 2 Analysis), Level 2 assigned one level lower rating than anticipated, e.g. Scour Condition Rating L2.ScCR 2 (Some damage). This will be looked at in more detail in paragraph below.

This case occurred for two bridges with internal Irish Rail reference UB01 and UB140. UB01 is a seven span bridge over river Liffey, just near the Heuston station in Dublin. UB140 is a fifteen span bridge on the River Barrow between Co. Kilkenny and Co. Waterford. The bridge is part of an old, decommissioned Rosslare-Waterford line.
This discrepancy can be explained if we look at the information about the foundation depths of these bridges. For both of the bridges, depth of foundations is known. Two methods (B1 and L2) use different rules for depth of foundations. Reports that rely on Method B1 assigned Priority Rating 3 – "Move to Stage 2 - Analysis" because the foundations are shallower than 3 times the maximum channel depth. This is a relative and sometimes unclear description. Method L2 has standardised rule(s) for calculation of components that take into account foundation depths, as defined in Table 6.11. Method L2 is superior to method B1 in this regard and the noted discrepancy is not of any concern.

The overall conclusion from the comparison is that for four bridges, e.g. (4%) the results using Method L2 gave one level higher or lower Scour Condition Rating than anticipated. Noted discrepancies were looked in detail and were justified as being acceptable.



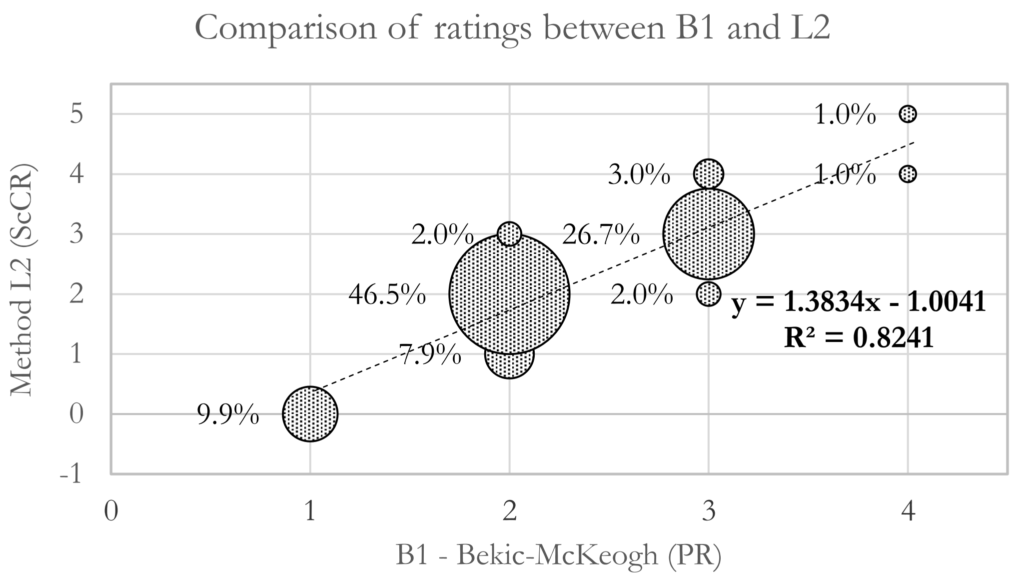


Figure 7.7 Results of the comparison between Method B1 and L2 for Data block 2.

7.4.3 Summary for the Correlation Analysis

Pair-wise comparison of correlation coefficients (see section 7.2) indicates very similar pattern. Strong correlation (R^2 = 0.82) is confirmed between methods B1, L1 and L2. Correlation combinations between other methods show weak fit, see coefficients of determination in Table 7.7.

A detailed analysis and calculation of correlation coefficients is shown in Annex M.

Table 7.7. Results of evaluation showing Coefficient of determination R^2.

Method	B1	L1	L2	C	B2a	B2b
B1	1.0	0.82	0.82	0.17	0.00	0.00
L1	0.82	1.0	0.75	0.20	0.00	0.00
L2	0.82	0.75	1.0	0.17	0.34	0.12
C	0.17	0.20	0.17	1.0	0.10	0.06
B2a	0.00	0.00	0.34	0.10	1.0	0.19
B2b	0.00	0.00	0.12	0.06	0.19	1.0

7.4.4 Conclusions for each method

7.4.4.1 Method B1

Method B1 was set as a basis for all of the comparisons. If the comparison with any of the methods with method B1 has a correlation coefficient close to 1, that method would be plausible for further application.

Also if the comparison between two methods shows that there are no unacceptable outcomes of the comparison between method X and method B1, method X would be considered desirable for further application.

7.4.4.2 Method L1

Method L1 shows strong correlation (R^2 = 0.82) with Method B1. There are no unacceptable outcomes from the comparison with Method B1. Method L1 is considered to be a good fit with method B1.

Method L1 is recommended for application for bridge scour inspections.

7.4.4.3 Method L2

Method L2 shows strong correlation (R^2 = 0.82) with Method B1. There is very low percentage of unacceptable outcomes (4%) from the comparison with Method B1. These outcomes are proven to be acceptable after detailed case-by-case analysis. Method L2 is considered to be a good fit with both methods B1 and L1.

Method L2 is recommended for application for bridge scour inspections.

As costing component is important for bridge management, Method L2 can be further improved by adding the costing component [6, 114] as one additional variable within Level 2 bridge scour assessment. From the point of view of safety to traffic over the bridge Method L2 is adequate for further application.

7.4.4.4 Method B2a

Priority Rating from Method B2a does not include the Severity of bridge Collapse and Furthermore, equations for calculation of potential scour depths tend to overestimate the extent of scour. This raises a doubt if methods B2a and B2b are reliable methods for application. Comparison of results shows that methods have unacceptable differences for 7 bridges (28%), which is in accordance with low coefficient of determination $R^2 = 0.19$.

When comparing the results of the L2 scour condition rating L2.ScCR with B2a Priority Rating, the one can observe a weak correlation ($R2 = 0.34$). In the conclusions (see section 4.3) it was noted that the weakest point in method B2a is calculation of scour depth, which is often overestimated when compared to the observations [143-145]. Further, only 4 (16%) of 25 bridges have information on foundations. Due to fact that the estimated scour depth in Method B2a is unreliable and the depth of foundations are unknown for 84% of bridges it is considered that the comparison, e.g. Method B2a is unreliable. Method L2 provides an option for more accurate whilst conservative estimation of foundation depths in case foundations are unknown. Therefore Method L2 can overpower Method B2a.

7.4.4.5 Method B2b

Qualitative Risk although very useful for managing of the overall costs does not account for actual scour depth. Therefore method is considered inadequate. However, Risk matrix can be introduced in order to further develop methods L1 and L2.

7.4.4.6 Method C

Report T112 [23] recommends enhancements to the Method C (EX2502) scour assessment procedure. Conclusion based on the application of Method C on 101 bridge in Ireland is that the method is inadequate for application as it has very week correlation with all other methods and it has a very large number of unacceptable outcomes when comparing to all other methods.

7.4.5 Overall conclusion from verification of new methods for bridge scour inspection

Methods L1 and L2 from new Inspection Module have been proven as reliable.

Both methods have passed sensitivity analysis, were tested on site and are applied and verified on 44 bridges (Method L1) and 101 bridges (Method L2) in Ireland.

In case that during their application some undesirable calculation of Scour Condition Rating is noted, both methods can easily be further adjusted by changing weighting factors in Method L1 or by adjusting lookup matrices for Method L2.

Methods L1 and L2 are recommended for an on-site application for the assistance of operable BMS. Weight factor(s) and lookup matrices are now verified. If the dataset with even larger sample(s) of the bridges become available, further adjustment is of the methodologies will be possible.

In the following chapter (Chapter 8) it will be explained how the planning of bridge inspections and results of the bridge scour inspections using new methods (L1 and L2) can be used effectively during or after flood events.

Chapter 8
Development of a flood forecasting system to assist management of bridge inspections

In this chapter a very effective way of combining flood forecasting and results of bridge inspections, e.g. bridge scour condition rating, will be outlined. This will be achieved by the incorporation of flood forecasting into the BMS.

The flood forecast will be used

- to plan and schedule bridge scour inspections, e.g. if the bridge should be inspected during low flows or post flood events.
- As input, in combination with Bridge Scour Rating for the Bridge Management System's DSS, in order to flag scour-susceptible bridges during and after flood events, flagging these bridges for re-inspection. In this way the bridge scour inspection interval can be scheduled in a more dynamic and therefore more effective way. The number of bridge scour inspections could even be reduced in a case where there were no flood events between two inspections.
- To develop a Real Time Scour Depth Model (SDM)

After the introductory section about general information and history of flood forecasting, a pilot site and its flood forecasting system developed during the writing of this thesis will be presented. The pilot site is located on river Bandon in Co. Cork in Ireland. The pilot site covers c. 600 km^2 catchment area.

For the pilot site, the flood forecasting system – Bandon FFS -- was developed during the writing of this thesis. A number of bridges located over river Bandon and Ilen were inspected using Methods L1 and L2. A concept and algorithm of how the inspection results can be used during flood events for more efficient inspections and management of flood events will be described. For one smaller bridge a real-time Scour Depth Model

(DM) was developed. Application of existing equations for estimation of scour depths in real time was not tried before.

A tool for estimation of set-up cost and running costs of a FFS was developed based on experience of setting up of Bandon FFS. The costing component will be elaborated in more detail in Chapter 9.

8.1 Introduction into the Flood Forecasting and Warning System(s)

Interest in Flood Early Warning Systems (FEWS) and development of Early Warning Systems (EWS) started in early 1970's in the USA (ALERT and IFLOW programmes); these programmes have been continuously developing and upgrading since 1970's [166]. In the early days the development of EWS was restricted mainly due to technology requirements (software, communication), insufficient data availability and data transfer, financial requirements, time and knowledge etc. Nowadays, advances in computational speed, Information and Communications Technology (ICT), cloud systems, data collection and measurement technology have reached the stage in which they might provide sufficient resources needed for the establishment of FEWS. These are some of the factors that have resulted in an increased number of operational and fully automated Flood Early Warning Systems (FEWS). Flood forecasting is becoming a well-recognised solution for flood management as an indirect measure for minimising the risk, should preventive or defence measures prove ineffective or are not feasible for implementation.

A Flood Forecasting System (FFS) involves the use of mathematical computer models and/or upstream-downstream gauge correlations (rain to flow rate or water level to water level correlation) to predict flood water levels based on actual meteorological data and tools. A Flood Early Warning System has the same function as FFS, with an additional function to disseminate flood forecasts to people at risk. The effectiveness of flood warning depends on detection, warning and response. Vogelbacher gave a good cross section of best practice in flood forecasting and warning [167].
FEWS can be implemented across countries (transnational), for large catchments (global) or for smaller catchments (local). Transnational and global FEWS have longer lead times, due to the larger areas involved, and provide more general output (less information), while

local FEWS have shorter lead times and provide more information (forecast flood maps, etc.). The development of every FEWS requires revision and comparison of forecasted and actual floods in order to improve performance and produce new concepts and models. A conceptual model of a FEWS consists of (1) Inputs, (2) data integrator / FEWS platform, (3) outputs (model results, warning and response) and (4) Revision / improvement (Figure 8.1).

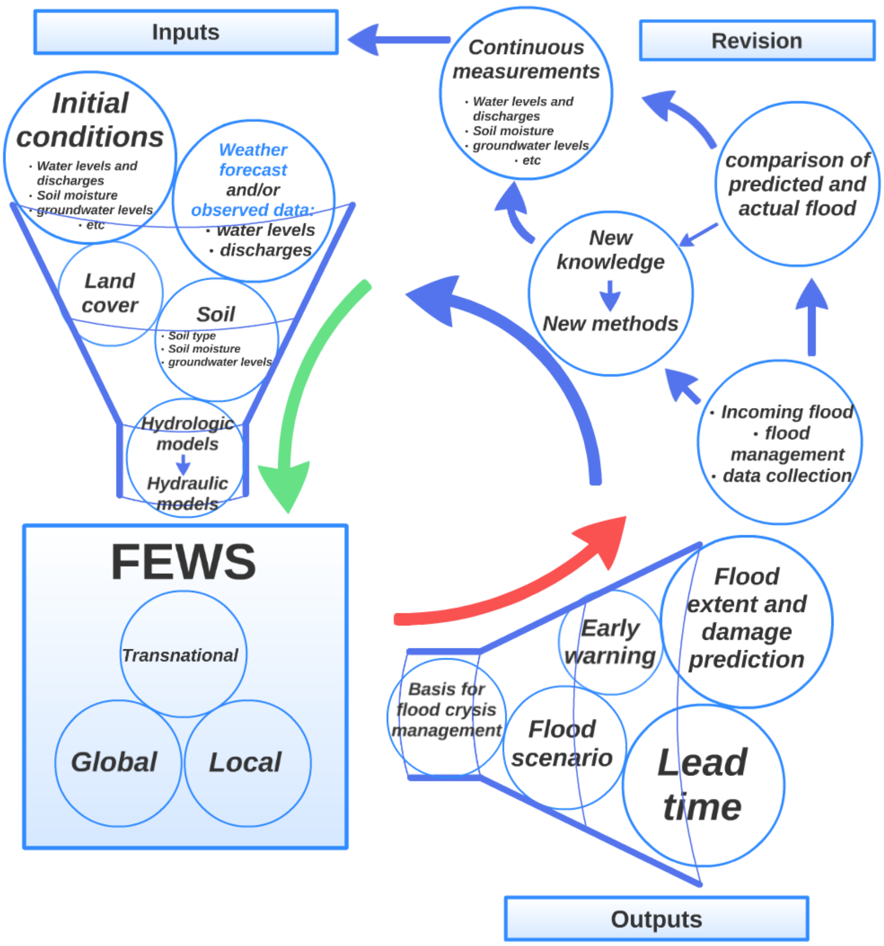


Figure 8.1. Conceptual model of modern FEWS.

The observed input data can be obtained from surface observations [167] (rainfall, air temperature and humidity, air pressure, soil moisture, etc.) or from space [168] (evaporation and soil moisture).

According to industry standards, two leading data integrator systems / platforms for Flood Forecasting can currently be identified. These are Delft FEWS by Deltares and Mike Operations by DHI. Ebel [169] in 2010 gave a cross section of Delft FEWS implementation, as shown in Figure 8.2. DHI's Selected References timetable of worldwide implementation between 1993 and 2012 is shown on the link below: (http://www.dhigroup.com/~/media/Publications/Solutions/RiversAndReservoirs/Selected%20References.ashx). The US Army Corps of Engineers HEC-RTS (Real Time Simulation) [170] data acquisition and hydrologic modelling system for short-term decision support of water control operations in real time can be outlined as a third competitor on the data integrator systems market.

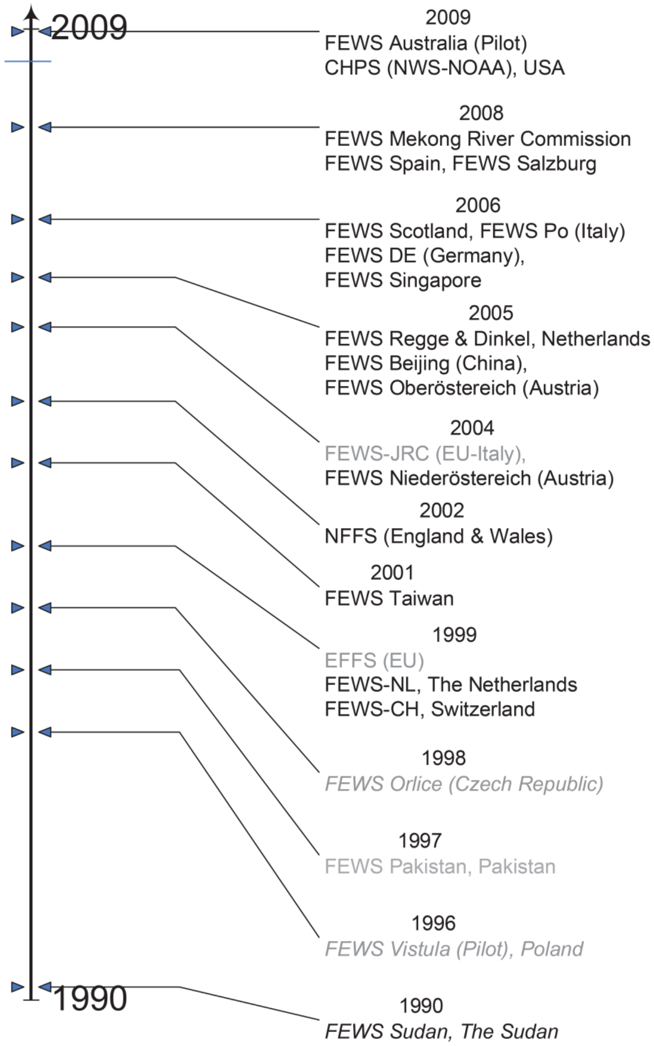


Figure 8.2. Timeline of FEWS Implementations from 1990 until 2009 (Grey indicates these are not operational) [169].

The results/outputs (water levels and flows) from the models integrated into the FEWS are useful only if they are compared to some site-specific thresholds. For sites with defined thresholds, a warning can be issued when thresholds are exceeded. The lead rime of the warning is very important in order to form an effective response (action) to a potential hazard. The sequential scheme of FEWS is shown in Figure 8.3 below.

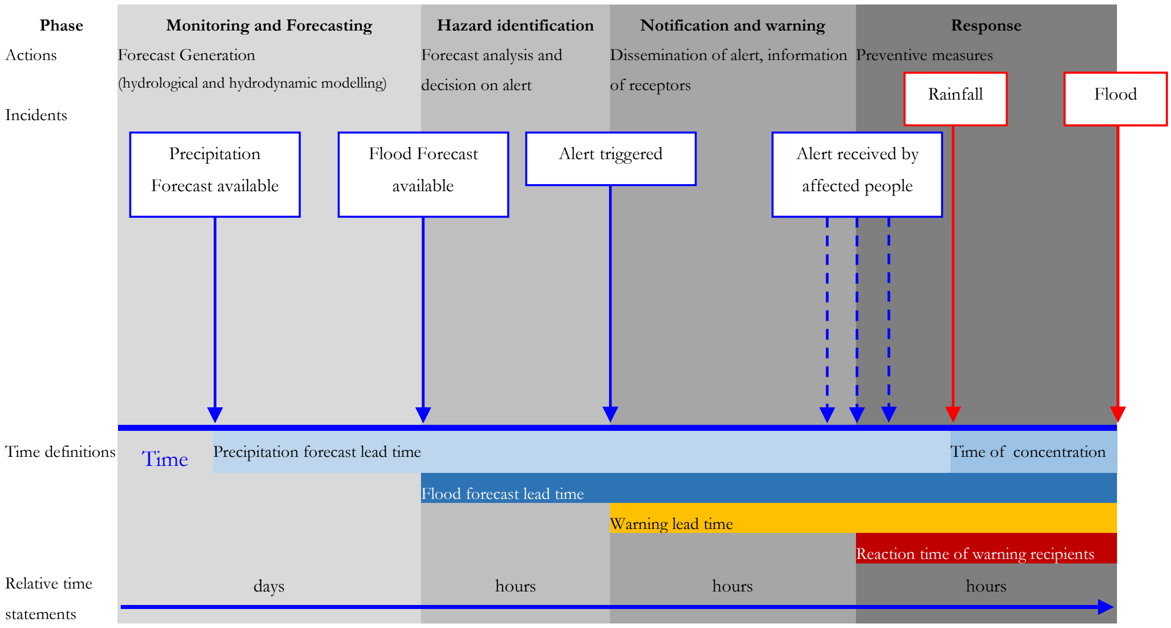


Figure 8.3 Sequential scheme of the Flood Early Warning System [171].

Guidelines on how to set up advanced warning systems and of risk communication strategies can be found in SUFRI project report [172]. The most challenging part of the process is deployment of monitoring networks, development and calibration of such hydrologic models. This means that data collection for the purpose of calibration of hydrological models extends the process of the FEWS development. Furthermore, in case there is no monitoring network prior to the decision on development of the FEWS, such systems should normally require development of 1D or 2D hydraulic models in order to obtain rating curves (curves that describe the relation between water levels and flow rates).

A discussion on benefits, effectiveness, advantages as well as disadvantages of FEWS will be provided in Chapter 9.

8.1.1 FEWS in Europe

European Flood Awareness System (EFAS), the first operational hydrological network in Europe, is operational since 2003. EFAS is developed to produce European overviews on on-going and forecasted floods up to 10 days in advance [173-175]. EFAS forecasted the Alpine floods of August 2005 in all river basins and reported in real-time. Since 2011, EFAS is part of the Copernicus emergency management service and has now been transferred to operational service in 2012 (http://www.efas.eu/). As part of this PhD work, collaboration with EFAS was initiated in August 2016 resulting in access to the EFAS products until the end of 2018.

In the SUFRI project [172] four case studies of FEWS are presented: Benaguasil and Arenys de Mar/Munt (Spain), Graz (Austria) and Dresden (Germany). In the section 8.1.1, meteorological forecast, models, lead times and output intervals for local FEWS that are used in Europe will be analysed. Different approaches can be used in order to establish FEWS (e.g. experiences with analogue warning systems, weather data, rainfall and flood forecast models, flood maps and flood management plans).

A screening through available meteorological data for FEWS in Europe will be listed in the following section 8.1.1.1.

8.1.1.1 Meteorological forecasts in use

In order to increase the lead time of the flood forecast, meteorological forecasts from the Numerical Weather Prediction (NWP) models are used as input into hydrological models. The meteorological forecasts for precipitation that are possible for application in Ireland are listed in Table 8.1.

Table 8.1 Meteorological forecast in Europe Applicable to Ireland.

Short Name	Long Name	Spatial Resolution	Temporal resolution	Runs [UTC]	Forecast	Provider	Website
ECMWF	ECMWF global model	16km	3h 6h (for 144h-240h forecast)	2/day: 00, 12	144h 240h	European Centre for Medium-Range Weather Forecasts	www.ecmwf.int
GFS	U.S. global model	28km	3h	4/day: 00, 06, 12, 18	240h	National Centers for Environmental Prediction (NCEP)	www.ncdc.noaa.gov
UKMO	North Atlantic European model	4km	3h	4/day: 00, 06, 12, 18	48h	United Kingdom Met Office	www.metoffice.gov.uk
HIRLAM	ECMWF High Resolution Limited Area Model	11km	3h	4/day: 00, 06, 12, 18	54 hours	Met Éireann	www.hirlam.org www.met.ie
HARMONIE		2.5km	1h	4/day: 00, 06, 12, 18	54 hours	Met Éireann	www.met.ie
COSMO-EU	Cosmo model	7km	1h	4/day: 00, 06, 12, 18	72	Deutscher Wetterdienst (DWD)	www.dwd.de
ICON-EU	ICOsahedral Nonhydrostatic Model	6.5km	1h	4/day: 00, 06, 12, 18	120h and 30h	Deutscher Wetterdienst (DWD)	www.dwd.de

8.1.1.1.1 *ECMWF*

The European Centre for Medium-Range Weather Forecasts is an independent international organisation set up, initially, as a research organisation to develop forecasting in the medium up to 15 days ahead. The ECMWF consists of 20 member States (Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Slovenia, Spain, Sweden, Switzerland, Turkey and United Kingdom) with broad range of supporting countries and institutions.

(a) ECMWF NWP model

Several forecasting suites are running operationally at ECMWF. Of interest to sailors, global 10-day forecasts are based on 00 and 12 UTC analyses. The forecast for the first 6 days has an output interval of 3 hours and the forecast beyond 6 days has a 6-hour output interval.

8.1.1.1.2 *GFS*

The Global Forecast System (GFS) is a global weather forecast model produced by the National Centers for Environmental Prediction (NCEP) in the US. Dozens of atmospheric and land-soil variables are available through this dataset, from temperatures, winds, and precipitation to soil moisture and atmospheric ozone concentration. The entire globe is covered by the GFS at a base horizontal resolution of 28 kilometres between grid points, which is used by the operational forecasters who predict weather out to 16 days in the future. Horizontal resolution drops to 70 kilometres between grid point for forecasts between one week and two weeks. The precipitation forecast applicable for Ireland has 15x28km grid with output interval of 3 hours. The forecast is issued 4 times daily at 00, 06, 12 and 18UTC [176, 177].

8.1.1.1.3 *UKMET (UKMO)*

The UK runs its global Numerical Weather prediction model four times a day. Runs based on 00 and 12 UTC are the main runs with the best data analyses. The runs at 06 and 18 UTC are for updating purposes. The global model uses a grid length of about 17 km. The global model provides forecasts up to 6 days ahead.

Starting with the global model, the Met Office also runs a North Atlantic European model with a 4 km grid length for 48 hours. The data from this are then used to run a Limited Area Model (LAM) just for the British Isles. This model is run to 36 hours only using a 1.5 km grid.

8.1.1.1.4 *HIRLAM*

The international research program High Resolution Limited Area Model (HIRLAM), covering Europe, was established in 1985 in the Nordic countries and today consists of the National Meteorological Services (NMSs) from 10 countries (Estonia, Finland, Iceland, Ireland, Netherlands, Norway, Spain, Sweden, Lithuania and France). Met Éireann (Ireland) runs a HiRLAM configuration on a domain covering much of Western Europe, the north Atlantic and eastern parts of Canada. 54 hour forecasts are produced four times a day on a 11km horizontal grid with 60 levels in the vertical. The output time interval of the forecast is 3 hours.

In 2005, the strategic decision was made for HIRLAM to engage in a close cooperation with the ALADIN consortium. Since then, the focus of the HIRLAM research collaboration has been on the convection permitting scale, and on adapting the AROME model [178] for use in the common ALADIN–HIRLAM NWP system, in order to make it accessible for all 26 countries. HIRLAM is became supressed by HARMONIE in operational use in Ireland.

8.1.1.1.5 *Aladin*

Aire Limitée Adaptation Dynamique Développement International (ALADIN) [179, 180] started in 1991 and consists today of 16 member countries (Algeria, Austria, Belgium, Bulgaria, Croatia, the Czech Republic, France, Hungary, Morocco, Poland, Portugal, Romania, Slovakia, Slovenia, Tunisia, and Turkey). Aladin is not directly applicable to Ireland, however collaboration between HIRLAM and Aladin resulted in the creation of the HARMONIE system that is now the main forecasting system for precipitation in Ireland.

8.1.1.1.6 *HARMONIE*

As described above, the HIRLAM adaptation for the AROME model [178] resulted in the creation of HIRLAM–ALADIN Research on Mesoscale Operational NWP in the Euromed (HARMONIE) script system.

Met Éireann runs a single operational Harmonie configuration on a domain covering Ireland, the United Kingdom and part of Northwest France. 54 hour forecasts are produced four times a day using on a 2.5km horizontal grid with 65 levels in the vertical. The output interval of the HARMONIE forecast is for Ireland is 1 hour.

8.1.1.1.7 *Cosmo-EU*

The Deutscher Wetterdienst (DWD) model COSMO-EU (COSMO Europe) covers the whole of Europe with 665 x 657 grid points and a horizontal grid spacing of 7 km. In the vertical there are 40 layers from the surface up to about 24 km above ground.

Model forecasts are computed eight times per day; based on the analyses at 00, 06, 12 and 18 UTC up to 78 hours, and based on the analysis at 03, 09, 15, and 21 UTC up to 30 hours.

8.1.1.1.8 *ICON-EU*

The DWD's regional ICON-EU nest within the ICON global model came into operation on 21.07.2015. There is a tightly coupled two-way interaction between the ICON-EU regional model and the global ICON. As the letters 'EU' suggest, the ICON-EU nest covers the whole of Europe.

The native model grid has a horizontal grid spacing of 6.5 km, the output grid a grid spacing of 0.0625° (~ 7 km). In the vertical, ICON-EU relies on 60 levels up to a height of 22.5 km.

The ICON-EU forecasts are available up to +120 hours from the four model runs at 00, 06, 12 and 18 UTC and up to +30 hours from the model runs at 03, 09, 15 and 21 UTC. The time interval for the forecast period up to +78 hours is one hour, the forecast periods between +81 and +120 hours are covered by a 3-hourly time interval. In the west and east, however, the nest's coverage extends far beyond the European territory, covering the area bounded by the coordinates 23.5°W–62.5°E, 29.5°N–70.5°N.

8.1.1.2 Models in use

8.1.1.2.1 *Correlation models based on observations*

This type of the statistical models has been used historically for medium to larger catchments. Once operational, they have been proven to be reliable and capable of predicting peak water levels and approximate time to peak, after the upstream observation is recorded. The advantage of this type of model is that they are based on observed data and are relatively easy and cost-effective to implement. The biggest disadvantage of such models is that it is not possible to establish a correlation without long term observations. This means that they can be implemented only on medium to large catchments for which gauges that offer reliable long-term hourly datasets are available.

An example of such a system is flood warning at the river Weißeritz in Dresden, Germany. The system is based on water level measurement at the flood information gauge in Freital which is situated about 10 km upstream. The flood propagation time between Freital and Dresden city is about 3.5 hours. Similar systems were historically implemented in Austria (Styria), Germany, Slovenia, Chech Republic and Croatia on the rivers Danube and Drava and in Slovenia, Croatia and Bosnia and Herzegovina on the rivers Kupa and Sava.

The decommissioned Bandon FEWS in Ireland is based on threshold river level alarms at monitoring stations upstream of Bandon town and issues warnings based on predefined Warning Codes.

8.1.1.2.2 *Hydrologic and Hydraulic models*

A short overview of the hydrological and hydraulic models and data integrators for different FEWS across the Europe is shown in Table 8.2 below.

Table 8.2 Models in use for FEWS in Europe.

FEWS in EU:
Hydrologic models
LISFLOOD (EFAS)
Rainfall/Runoff model NAM (Austria: FFS Raab, Mur, Slovenia: BOBER on Sava and Soča, Mura)
Kalypso (Germany: Alster)
X-Nash (Italy: Umbria)
Mike Drift (Italy: Umbria)
HEC-HMS (Sava basin, Italy: Umbria, River Mura in Croatia)
Stafom (Italy: Umbria)
Hydraulic models
Mike 11 (Austria: FFS Raab, Mur, Slovenia: BOBER on Sava and Soča, Mur, Germany: Alster, Italy: Umbria, Sava basin)
MikeFLOOD (Gleisdorf on Raab river in Austria)
HEC-RAS (Germany: Alster, Italy: Umbria)
HEC-RAS (Sava basin)
Real-time data and forecast modelling tools
MikeFLOOD WATCH (Austria: FFS Raab, Mur, Slovenia: BOBER on Sava and Soča, Mur)
Delft-FEWS (Scotland, Italy, Netherlands, Switzerland, Austria, Ireland, Sava basin, etc.)

In order to determine the appropriate model for the new Bandon FFS focus will be made on hydrological (rainfall-runoff) models. There are numerous hydrological (RR) models that are in use. The main categorisation of hydrology models is between lumped (semi-distributed) and distributed models. The first conceptual hydrological models were lumped models due to the limited computational resources available, the lack of spatial description of catchment characteristics and the limited rainfall records at points. Due to increased availability of radar rainfall measurements, distributed precipitation model forecasts, availability of digital terrain models (DTM) on grids from 10m to 50m, land covers, soil types, etc. distributed models now have a greater role.

An overview of the Lumped and Distributed hydrological models is shown in Table 8.3. Devia et al. [181] gave a review of a VIC, TOPMODEL, HBV, MIKESHE and SWAT models. A utilisation of a tRIBS model on a catchment in Ireland was analysed by Steinmann [182].

Table 8.3 Overview of the hydrological Lumped and Distributed models

No	Model ACRONYM	Type of model	Link
1	HEC-HMS	Lumped	http://www.hec.usace.army.mil/software/hec-hms/
2	EPA SWMM	Lumped	https://www.epa.gov/water-research/storm-water-management-model-swmm
3	APEX	Lumped	http://www.geo.uzh.ch/en/units/h2k/services/hbv-model/
4	ARNO	Lumped	http://www.idrologiaeambiente.it/amministrazione/cms/files/71/ARNO.pdf
5	HBV/IHMS	Lumped	http://www.smhi.se/forskning/forskningsomraden/hydrologi/hbv-1.1566
6	HYPE	Lumped	http://www.smhi.se/en/research/research-departments/hydrology/hype-1.7994
7	Mike 11 NAM	Lumped	https://www.mikepoweredbydhi.com/download/mike-by-dhi-2014/mike-11?ref=%7BCF5835F0-51C9-46F3-8134-57BE71954D19%7D
8	SOBEK 3 D-RR	Lumped	https://publicwiki.deltares.nl/display/nghs/SOBEK+3
9	HYDROFOSS	Lumped	https://grasswiki.osgeo.org/wiki/AddOns/GRASS_6#HydroFOSS
10	IHACRES	Lumped	http://www.toolkit.net.au/tools/IHACRES
11	TOPMODEL	Lumped	https://grass.osgeo.org/grass70/manuals/r.topmodel.html
12	SWAT	Lumped	http://swat.tamu.edu/software/qswat/
13	WFLOW	Distributed	http://wflow.readthedocs.io/en/latest/
14	LisFLOOD	Distributed	https://ec.europa.eu/jrc/en/publication/eur-scientific-and-technical-research-reports/lisflood-distributed-water-balance-and-flood-simulation-model-revised-user-manual-2013
15	Mike SHE	Distributed	https://www.mikepoweredbydhi.com/products/mike-she
16	tRIBS	Distributed	http://vivoni.asu.edu/tribs.html

8.1.1.2.3 *Lead times of observed FEWS*

The following lead times were identified for different systems in the Europe:

- <3 hours: Benaguasil (45-60 min), Arenys de Mar/Munt (35 min)
- ≥3 hours: Dresden (3.5 hours), Bandon in Ireland (5 hours), Umbria IN Italy (10 hours)
- ≥3 days: UK Environmental Agency
- ≥6 days: EFAS, FFS Austria on Mur, BOBER in Slovenia

In the report [183] an effectiveness (items that can be protected) of a lead time are analysed. When lead times of some FEWS listed above are compared to the Table 8.4 it can be seen that the lead time for some of the above FEWS is insufficient.

Table 8.4. Items Protected with Warning [183].

<30 min warning	<2 hours warning	<4 hours warning	>4 hours warning
Television	Karaoke	Large appliances (e.g. refrigerator)	Big oven, freezer
Stereo equipment	Microwaves, small stove, toaster	Bookcases, dining tables, chairs	Kitchen utensils
Small electrical appliances	Items in cupboards	Carpets	Beds
Personal effects	Expensive clothing	Additional clothing and personal effects	
	Vehicles	Chicken, pigs	

8.1.1.3 Local FEWS in Ireland

A relatively small FEWS in Ireland, the Bandon Flood Early Warning System, (http://www.bandonfloodwarning.ie), started operating in 2011, and was decommissioned in 2019,. The system is based on the observations of water levels on the water levels from three gauges upstream of town Bandon, which can be correlated to the water levels in Bandon town. This allows Cork County Council to give up to 5 hours prior warning of a flood event. The message to the residences of Brandon town is transferred over SMS.

Other catchments that recently developed or are currently under the development use Delft FEWS as data integrator. The first is River Suir at Clonmel system developed for Tipperary County Council which is a fully operational system, based on the URBS and WFLOW hydrological model developed by OPW and Deltares respectively. The system is mainly used as a decision support for deployment of the river Suir flood Barriers.

The systems in Ireland that are currently under development are Blackwater river, Lee river, Bandon river and Ilen-Caol (Skibbereen) river systems. The details of Bandon FFS will be analysed in more detail as its development is part of this PhD work. In the following section a rationale for the inclusion of flood forecasting system (FFS) into BMS will be explained.

8.2 Development of Bandon FFS

8.2.1 Schematisation of the monitoring and prediction module

The monitoring and prediction module, e.g. Flood Forecasting System (FFS) consists of monitoring network (observations) and hydrological model(s), which transforms observed rainfall into the runoff (flow rate). The schematics of the monitoring and prediction modules is showed in a flow chart (Figure 8.4).

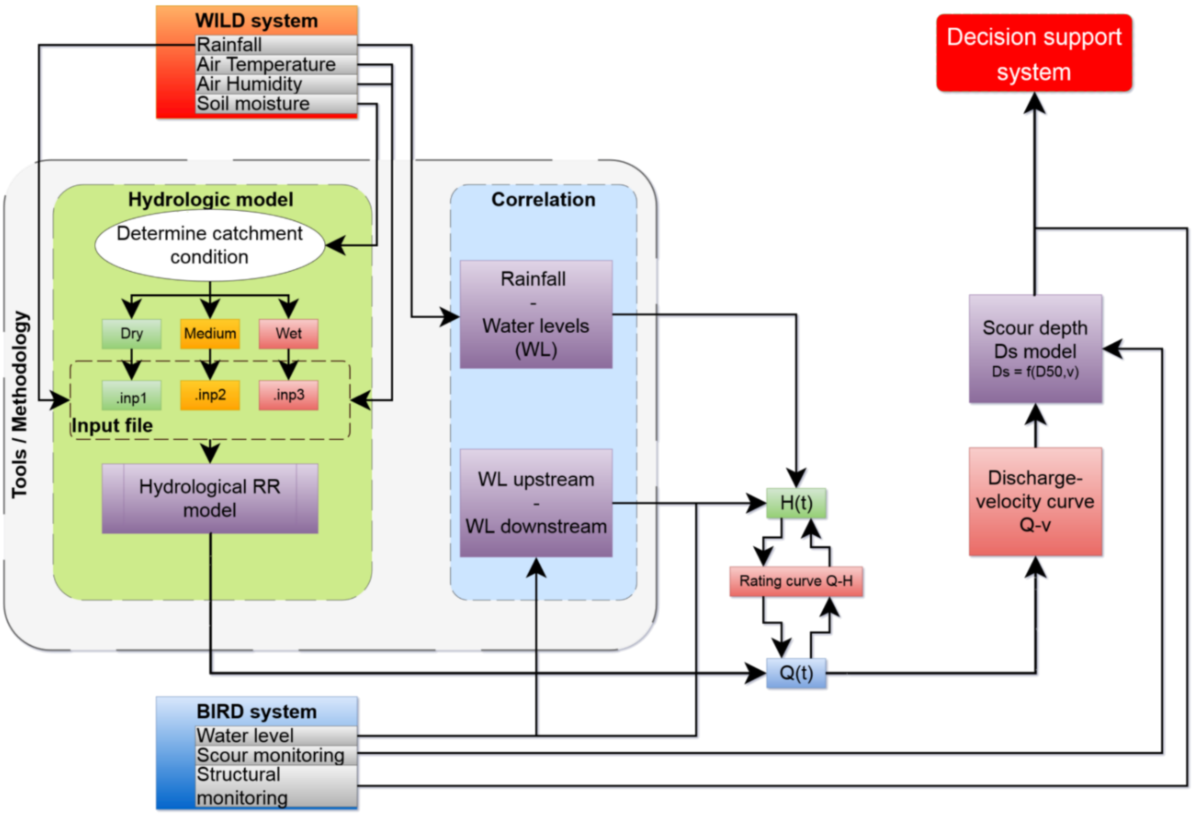


Figure 8.4. Flow chart of WILD BIRD systems which are employed to provide real-time environmental and structural input data and assess the potential of flood hazards in the catchment and bridge site areas.

The installed monitoring network includes meteorological observations - WILD[11] (precipitation, air temperature and humidity, soil moisture) and hydrologic observations - BIRD[12] (water levels and flow rates).

The meteorological observations are used as input data for hydrological modelling and now-casting resulting in flow hydrographs at the bridge including flow rates and water level information. Soil moisture and rainfall data is used to determine the appropriate hydrological model set-up, which consists of either (a) dry catchment conditions, (b) medium catchment conditions or (c) saturated catchment conditions.

When the hydrological model set-up is determined, observed rainfall is transformed into the effective (nett) rainfall. The output from the hydrological model is a now-cast flow hydrograph, discharge $Q(t)$ with a lag time up to 24 hours. $Q(t)$ is correlated to water levels $H(t)$ using an existing rating curve (Q-H) and to the flow velocity $v(t)$ using existing discharge-flow velocity curve (Q-v).

The role of hydrological observations is to provide real-time information of hydrologic conditions at the bridge and for verification and improvement of the prediction module (comparison of predicted and observed water levels and flow rates).

[11] Weather Information Logging Device instrumentation developed within Bridge SMS consortium
[12] Bridge Information Recording Device instrumentation developed within Bridge SMS consortium

8.2.2 Monitoring Module

8.2.2.1 Components of the monitoring network

As part of an environmental monitoring station, there are four key modules [111] which must be combined so as to provide real time monitoring, namely:

1) Sensors – It is required that each environmental element being monitored has the appropriate sensor to accurately monitor it. Such sensors range in price and accuracy, but for current monitoring stations sensors which conform to standards as established by legislation, the mechanisms are consistent, e.g. standards established for monitoring rainfall using a tipping bucket mechanism with uniform volume requirements of the bucket size and the accuracy of the readings.
2) Data-logging – There exists wide variation in available data-logging modules, in terms of number of sensor inputs available for logging, input types, programming requirements, reliability, cost and power consumption. While many cost effective solutions are available, such solutions are often not appropriate for remote monitoring in harsh environments nor are the connecting and programming requirements for integrating the sensors user-friendly.
3) Telemetry – As with the data-logging module, a diverse range of telemetry units exists, which can be used with remote environmental monitoring stations, in terms of cost, reliability, programming, power consumption and connection to data-logging module.
4) Power Supply – When individual modules are integrated, the power demands for the entire station can be such that a connection to the mains power is required or a significant renewable source that is capable of satisfying the power needs for real-time monitoring. Significant cost and expertise is therefore required to ensure that the power requirements of the station are met and that the connections to the sensors, data-loggers and telemetry modules are established correctly.

8.2.2.2 Screening of the existing monitoring network

After selection of the pilot catchment for the demonstration of Flood Forecasting system within Bridge Management System, detailed screening of the existing monitoring network was initiated. As stated above, Bandon Catchment was selected as a pilot study which would introduce Flood Forecasting into a Bridge Management System.

The screening of the existing monitoring network on Bandon Catchment showed that:

- The existing network of meteorological observations consists of a single gauge providing rainfall observations every 15 minutes. The station is located in Dunmanway, it is operated by RPS and the data is available periodically upon request.
- existing network of hydrological observations consists of five (5) hydrological stations, operated by Cork County Council and Office of Public Works which enable real time observations of water levels every 15 minutes.

Based on the conclusions from the screening of existing monitoring network it was concluded that there is a need for extension of existing monitoring network. It was decided to expand the existing monitoring network with an additional two meteorological stations (distributed over the catchment) and two hydrological stations (positioned at the location of pilot bridges). Market research on data loggers was conducted in order to acquire the most cost-effective data loggers.

Table 8.5. Comparison of Market pricing of the data loggers and telemetry systems.

Purchase Model	Telemetry Module	Data-Logger Module	Total Price
Commercially available separate systems	€150 - €420	€570 - €1300	€720 - €1720
Commercially available integrated systems	n/a	n/a	€800

NOTE: Overall Price would need to include cost of sensors (€500-€1000), Power Source, Battery system, installation and maintenance (running of servers) costs, etc.

Development and deployment of monitoring equipment is described in section 8.2.2.3 below.

8.2.2.3 Development of Monitoring module

The installed Monitoring Network used for the prediction module consists of existing sensors and newly deployed devices (W - WILD and B - BIRD), as shown in Figure 8.5. Meteorological observations (green dots in Figure 8.5) represent wider area of the river catchment. Hydrologic observations (blue triangles in Figure 8.5) provide information on water levels and flow rates for a single location such as Bridge.

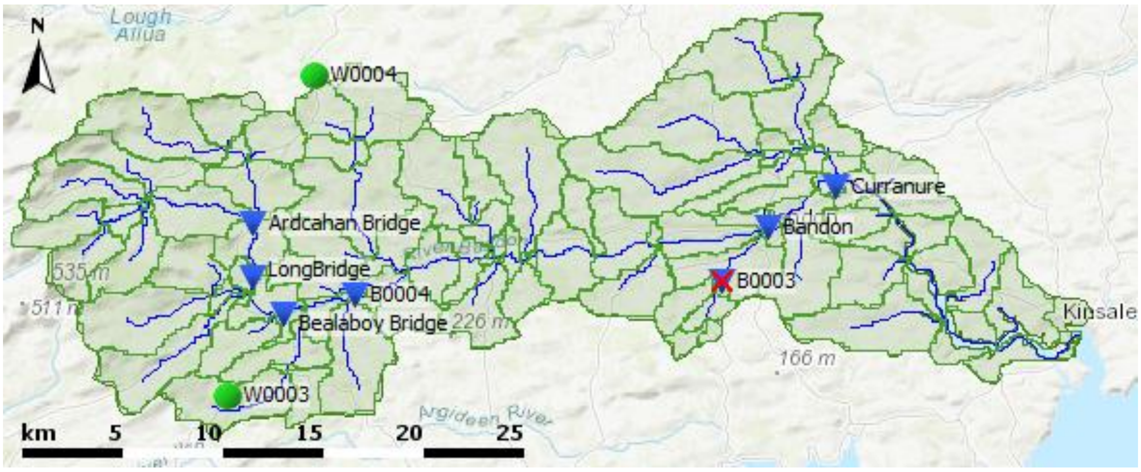


Figure 8.5. Installed monitoring network on Bandon Catchment (up to date).

WILD and BIRD systems complement each other, e.g. water level monitoring information can be used for verification of now-casting hydrographs obtained from the hydrological modelling but also to provide an accurate real-time information of water fluctuations at the bridge site. Water levels and flow rates are correlated to the flow velocity which also provide the basis for the prediction of scour depth calculation.

Two BIRD devices were deployed at Bandon catchment at Meelon (B0003) and Manch bridge (B0004) on 28th August 2017. The locations of the BIRD devices are shown in Figure 8.5. Schematics for the installation is shown in Figure 8.6. The housing for the BIRD system has been provided using double plastic casing - under IP67 and IP56 standards. The photographs from the installation of BIRD devices are shown in Figure 8.7.

8.2.2.3.1 *WILD*

Weather Information Logging Device (WILD) is a newly deployed data logging and telemetry system using commercially available sensors for measurement of precipitation, soil moisture, air temperature and humidity. The system requires power from the electricity distribution network.

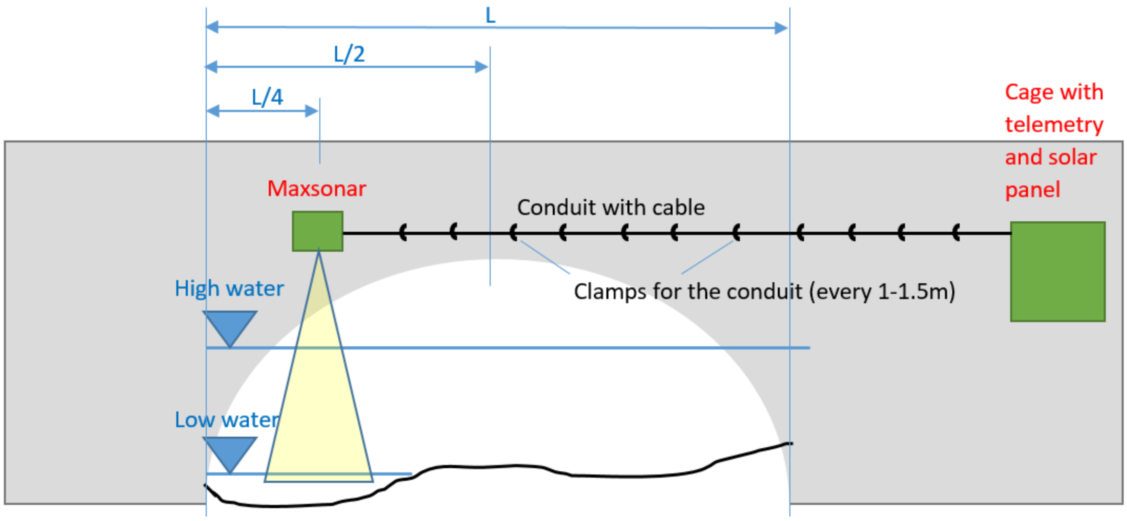
The purpose of the device is to provide information relevant for a catchment (an area of land where water collects when it rains, often bounded by hills, flows over the landscape and infiltrates in the soil eventually feeding the river).

Rain gauge data recorded continuously and a sum of the rainfall data is stored and it is sent to a server every 15 minutes. Real time air temperature and humidity and soil moisture data is stored to a SD card every 15 minutes and sent to designated buffer servers. For higher data security and preventing the loss of data two buffer servers were established.

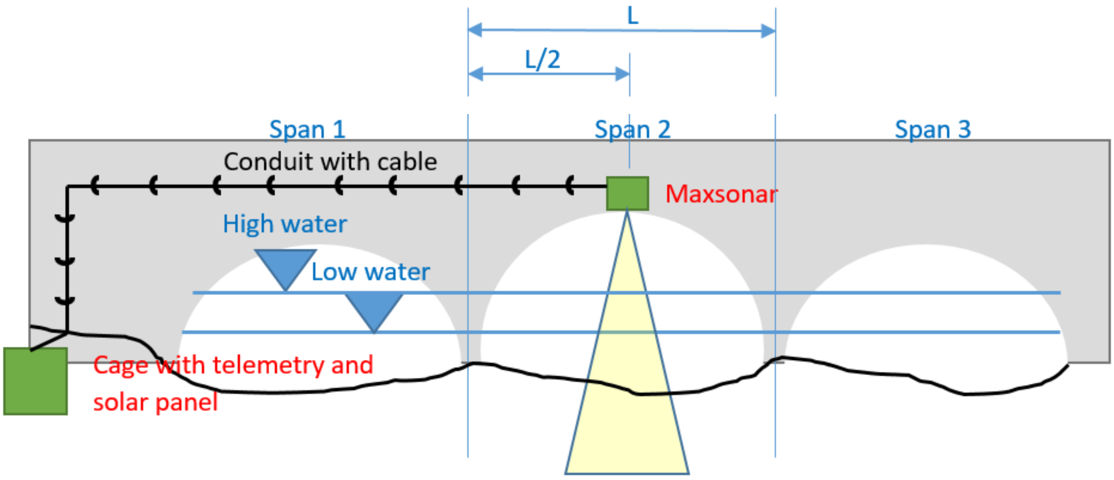
8.2.2.3.2 *BIRD*

Bridge Information Recording Device (BIRD) comprises of data logger and telemetry system and sonar sensors for contactless measurements of water levels in Malin Ordinance Datum. Based on the rating curve (relation of water levels and flow rates), water levels are translated into flow rates Q(t). The newly developed BIRD system integrates all four key modules of an environmental station. The data logger and telemetry is based on an Arduino platform, as for the WILD device. The device autonomy depends on a battery which is charged with a solar panel, meaning that the system is self-sufficient. The power is obtained using a monocrystalline 30W-12V solar panel, charging chip (TP4056) with protection circuit (MOSFET and battery protection chip) and a 12V battery (Geltech accumulator 12/8 Ah).

When the water levels reach assigned thresholds, data is transmitted every 15 minutes. The conditions for data transmission of the two BIRD devices, shown in Table 8.6, are defined in order to lower power consumption during the conditions of low water levels.



a) Meelon Bridge (B0003)



b) Manch Bridge (B0004)

Figure 8.6. BIRD installation schematics.

Data logger, telemetry, solar panel and battery

Sonar device

a) Meelon Bridge (B0003)

Data logger, telemetry, solar panel and battery

Sonar device

b) Manch Bridge (B0004)

Figure 8.7. BIRD installation (28th Aug 2017).

Table 8.6. BIRD transmission conditions.

Meelon	**REF = 51.16 [mOD]**				
Filter	**Time interval ti [min]**	**Distance for low waters Dlw [mm]**	**Change of distance in time interval ti ΔDti [mm]**	**Condition**	**Action**
A) Pre-Filter	ti = t15 = zadnjih 15 [min]	N/A	ΔDti = 500 [mm]	If D>=1555 [mm] And If |ΔD(t15)| >= 500 [mm]	Then = do not store the reading on SD card and do not transmit the reading
B) High Water levels	ti = t120 = last 120 [min]	Dlw = 1555 [mm]	ΔDti = 50 [mm]	If D<1555 [mm] And If |ΔD(t120)| > 50 [mm]	Then = transmit the readings to a buffer server every 15 [min] and store readings on the SD card Else = store readings on the SD card
C) Backup	ti = 12h	N/A	N/A	N/A	Bi-daily at 12:00h and 24:00h transmit all data from SD card to a buffer servers

Manch	**REF = 51.16 [mOD]**				
Filter	**Time interval ti [min]**	**Distance for low waters Dlw [mm]**	**Change of distance in time interval ti ΔDti [mm]**	**Condition**	**Action**
A) Pre-Filter	ti = t15 = last 15 [min]	N/A	ΔDti = 500 [mm]	If D>=4000 [mm] And If |ΔD(t15)| >= 500 [mm]	Then = do not store the reading on SD card and do not transmit the reading
B) High Water levels	ti = t120 = last 120 [min]	Dlw = 4000 [mm]	ΔDti = 75 [mm]	If D<4000 [mm] And If |ΔD(t120)| > 75 [mm]	Then = transmit the readings to a buffer server every 15 [min] and store readings on the SD card Else = store readings on the SD card
C) Backup	ti = 12h	N/A	N/A	N/A	Bi-daily at 12:00h and 24:00h transmit all data from SD card to a buffer servers

Where, *D [mm] = Distance from sensor*
Dlw [mm] = Distance for low waters
ti [min] = Time interval before the last sensor reading
ΔDti [mm] = Change of distance in time interval ti
REF [mOD] = Reference level
H [mOD] = Water level = REF - (D/1000)

8.2.2.4 Output of Monitoring module - Input rainfall for Prediction module

The input rainfall dataset for each forecast is 24-day rainfall which consists of the:

(1) 14 day recorded rainfall from instrumentation deployed over a catchment (WILD 1 and WILD 2);

(2) 52 hours rainfall HARMONIE NWP forecast issued by MetÉireann (forecast is issued every 6 hours with a time step of 1 hour and typical grid cell size of 2.5km); and

(3) 10 days rainfall NWP forecast from Global Forecast System (GFS) issued by National Centers for Environmental Prediction (NCEP) every 6 hours (00, 06, 12 and 18UTC) with a time step of 3 hours and relatively coarse grid cell size of 28km.

Two meteorological rainfall forecasts were elaborated in section 8.1.1.1. The schematics and example of input rainfall for 27th December 2017 is shown in Figure 8.8.

a) Input rainfall schematics

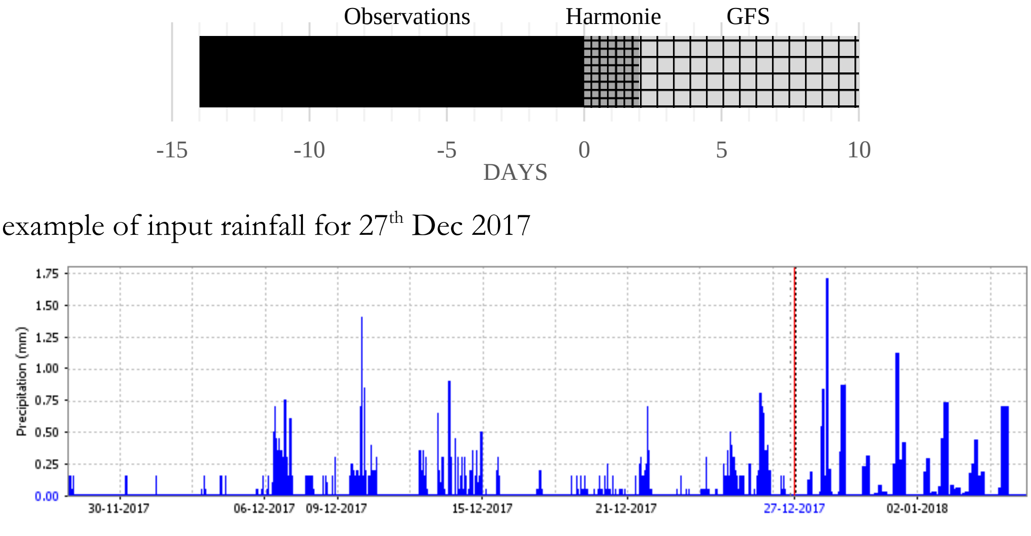


Figure 8.8 Input rainfall for the hydrologic forecast (note that "0 days" represents the date and time when the simulation is computed).

8.2.3 Prediction model

The prediction model consists of the data integrator system which enables validation and transformation of data from sensors and database in a way that is readable by hydraulic or hydrologic models.

Bandon FFS uses the hydrological rainfall-runoff HEC-HMS model (version 4.2.1) [184] for the calculation of the effective rainfall and output hydrographs. The HEC-HMS was selected due to its simplicity that enables standardisation of model set-up process and high accuracy that was proven in the calibration process (Annex N). HEC-HMS is a lumped model that requires less computational power, it supports SCS method which enables to use rainfall as only variable input over time. Razmkhah [185] found that the SCS Method is the second most suitable method in stream flow simulation. Thus, the system relies on only one sensor type. Furthermore, the model set-up process requires soil maps and land use data that is available online under open-source licence. The model set-up process is straightforward and transferable to other catchments and systems across the Europe and worldwide.

The HEC-HMS model was developed as part of this thesis during the BRIDGE SMS project (see www.bridgesms.eu).

The results from the hydrologic model, e.g. flow hydrographs, will be processed and analysed by data integrator. The data operator can perform the following operation(s) that will be applied within the BMS:

- Select the official forecast using API index
- Interpolate input rainfall (from observations and meteorological forecast) into sub-catchment polygons
- Highlight the bridges under flooding, based on the pre-defined flood water level thresholds
- Reads Scour Condition Rating of the bridge and creation of the list of bridges that need to be inspected after flood event
- Transform flow rates outputs at the location of the bridge into the scour depth (SDM)

The data integrator system used for Bandon FFS is Delft Flood Early Warning System (Delft FEWS). All elements of the prediction module and their development will be explained in the sections below.

8.2.3.1 Data integrator system

Delft-FEWS [186-188] is an open data handling platform initially developed as a hydrological forecasting and warning system. Essentially it is a sophisticated collection of modules designed for building a hydrological forecasting system customised to the specific requirements of an individual organisation. Because of its unique characteristics concerning data importing and processing and model connections, Delft-FEWS has also been applied in a wide range of different operational situations. Examples are water quality forecasting, reservoir management, operational sewer management optimization, and even peat fire prediction.

Delft-FEWS offers many options for the user to interact with the system. For a modern operational (forecasting) system this interaction is crucial. In water management, and other sectors, different types of models are being used to simulate real-world processes. Delft-FEWS is capable to connect to many of these models, and new connections can be made easily.

Delft-FEWS offers numerous options for interoperability and (model)interaction.

- The Delft-FEWS platform connects easily to a large range of hydrologic, hydraulic, and groundwater models. Actually, any program that uses or provides data can be connected. A model adapter forms the 'interface' between Delft-FEWS and a (forecasting) model of any model supplier.
- Delft-FEWS supports over 175 import formats and is able to export data in more than 60 export formats.
- Within Delft-FEWS an Application Programming Interface (API) will be provided to allow clients and consultants to develop their own Java plugins for imports, exports, displays, statistical functions and transformations (more information).

- Delft-FEWS provides easy to understand, advanced graphical and map-based displays. Forecast results can be disseminated through configurable HTML formatted reports, allowing easy communication to relevant authorities and public through intranet and internet. Standard output formats such as HTML formatted reports are available, and can be easily customised to specific user requirements.

8.2.3.2 Hydrological model

8.2.3.2.1 *Theoretical background for hydrologic model*

HEC-HMS is a lumped model designed to simulate the precipitation-runoff processes of dendritic watershed systems. It is designed for a wide range of geographical areas such as large river basin water supply and flood hydrology and small urban or natural watershed runoff, scour countermeasure design and flood forecasting. Long term hydrologic simulation of SCS-CN model was conducted by Geetha et al [189]. A typical representation of catchment runoff is shown in Figure 8.9.

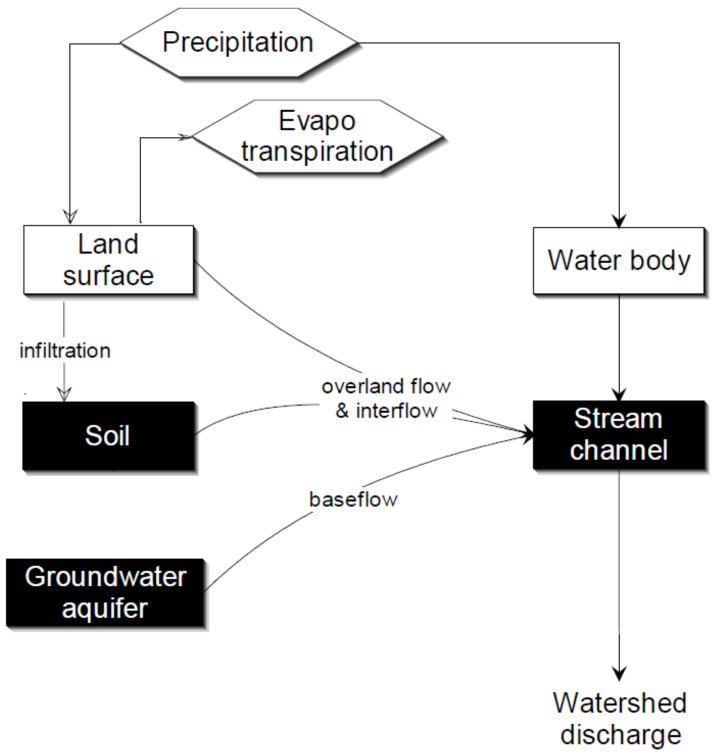


Figure 8.9 A typical representation of catchment runoff within HEC-HMS [184].

A HEC-HMS software package integrates different models for calculation of effective rainfall, rainfall to runoff transform methods, baseflow methods, routing methods and input design storm as shown in Table 8.7.

Table 8.7 Calculation models integrated into HEC-HMS software package

HYDROLOGIC ELEMENT	Calculation model	MATHEMATICAL MODELS
Catchment Model Subcatchment	A) Loss rate (Effective rainfall)	**Selected: SCS Curve Number** Other available: Deficit and constant, Exponential, Green and Ampt, Smith Parlange, Soil moisture accounting
	B) Rainfall to runoff Transformation	**Selected: SCS Unit Hydrograph** Other available: Clark UH, Kinematic wave, ModClark, Snyder UH, User specified S-graph, User Specified UH
	C) Base flow	Not used. Constant monthly, Bounded recession, Linear reservoir, Nonlinear Boussinesq, Recession
Stream Reach	D) Routing method	**Selected: Kinematic Wave** Other available: Lag, Modified Plus, Muskingum, Muskingum-Cunge, Straddle stagger
Meteorological model	E) Design storm (input rainfall)	**In use: Specified Hyetograph**, Other available: Gridded precipitation (MetEireann, GFS)

Performance of loss methods in HEC-RAS was analysed by Zerma et al. [190] (SCS CN, Green-Ampt (G.A.) and Initial and Constant (I.C.)) and Razmkhah [185] (Conservation Service (SCS CN), Green and Ampt (G.A.), Initial and Constant (I.C.), Deficit Constant (D.C.), Constant Fraction (C.F.), exponential (Exp.) and Soil Moisture Accounting (SMA)). Razmkhah [185] found that the SMA method was the first and SCS and Exp were placed as the second most suitable methods in stream flow simulation. SCS is placed as the second-best method in predicting the peak flow, but the worst at predicting volumes. Zerma et al. [190] found that the SCS CN loos method integrated within HEC-HMS model gives the best results overall.

In the sections below, selected calculation models for the effective rain, rainfall to runoff transformation and routing method are explained in more detail. The input rainfall is described in section 8.2.2.4.

(a) Loos method for calculation of effective rain

Based on the analysis above, and available input data in real time, the Soil Conservation Service (SCS) Curve Number (CN) model [184, 191] from 1972 is selected for the loos method. The model estimates precipitation excess as a function of cumulative precipitation, soil cover, land use and antecedent moisture.

The total precipitation P [mm] equals, see Figure 8.10:

$$P = P_e + I_a + F_a \qquad \text{(eqn 8.1)}$$

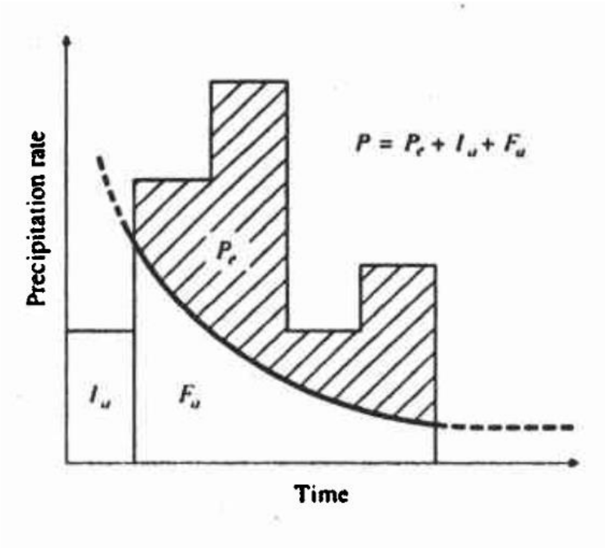


Figure 8.10 Variables in the SCS method of rainfall abstractions [191].

The principal of the SCS method is that the ratio of the two actual potentials quantities of precipitation are equal, see (eqn 8.2):

$$\frac{F_a}{S} = \frac{P_e}{P - I_a} \qquad \text{(eqn 8.2)}$$

where,

P_e - The depth of excess or effective rainfall runoff [mm]

I_a - Initial abstraction (initial loss) before pounding [mm]

S - Some maximum retention [mm]

F_a - Additional depth of water retained in the catchment after runoff after runoff begins [m]. It is equal or less then maximum retention S.

An empirical relation between initial abstraction Ia and maximum retention S is defined as with (eqn 8.3):

$$I_a = 0.2S \qquad \text{(eqn 8.3)}$$

By combining equations (eqn 8.1)-(eqn 8.3), the effective rainfall equals:

$$P_e = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \text{(eqn 8.4)}$$

The maximum retention is calculated based on the Curve Number (CN) as:

$$S = \frac{25400 - 254CN}{CN} \qquad \text{(eqn 8.5)}$$

The calculation of Curve numbers (CN II) will be described in section 8.2.3.2.4.

(b) Transformation of rain to runoff

The transformation of the of effective rainfall is defined by SCS unit hydrograph [184, 191], see (eqn 8.6). SCS unit hydrograph is dimensionless, single-peaked UH that corresponds to a runoff of 1mm of effective rainfall. SCS UH is transformed to a run-off hydrograph based on the peak discharge (*Q_{max}*) and rising time to the Peak of the hydrograph (*Tp*), see Figure 8.11.

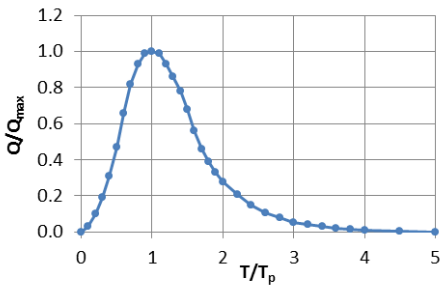


Figure 8.11. SCS Unit Hydrograph

$$Q_{max} = C\frac{A}{T_P} \quad \text{(eqn 8.6)}$$

Where,

A - watershed area [km^2]

C - conversion constant (2.08 in SI)

T_{lag} - time difference between centre of mass of rainfall excess and peak of UH [min]

T_p - Time of Peak [min]

$$T_P = \frac{\Delta t}{2} + t_{lag} \quad \text{(eqn 8.7)}$$

(c) Routing method

The kinematic wave routing method [184, 191] for calculation of the transformation of hydrographs within river network is used. The method uses the fundamental equations of open channel flow: momentum equation and continuity equations.

One-dimensional momentum equations is shown in (eqn 8.8) and continuity equation is shown in (eqn 8.9).

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \quad \text{(eqn 8.8)}$$

$$Q = \frac{CR^{2/3}S_f^{1/2}}{N}A \quad \text{(eqn 8.9)}$$

Where,

S_f – energy gradient (friction slope)
S_0 – Bottom slope
V – velocity
y – hydraulic depth
x – distance along the flow path
t – time
Q – flow rate
R – hydraulic radius
A – cross section area
N – resistance factor
n – Manning roughness coefficient

8.2.3.2.2 *Model description*

A lumped hydrological HEC-HMS (US Army Corps of Engineers) model of Bandon Catchment was used for flood forecast. The model transforms rainfall into a runoff for 94 sub-catchments (covering a total area of 591km^2) and routes flow through 127.6km river network, giving detailed flow rates at 58 point locations (bridges and river junctions). The model layout with location of the key control points is shown in Figure 8.12.

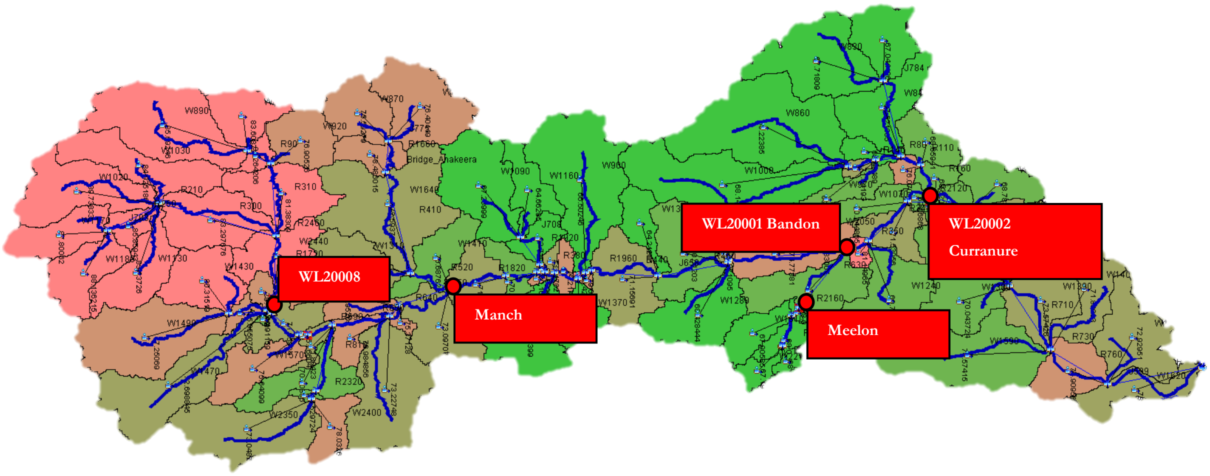


Figure 8.12. Hydrological HEC-HMS model used in hydrological forecast.

The model calculates effective rainfall using SCS Curve Number. The rainfall is then transformed for every sub-catchment (no. 94) into runoff (flow rate) using the SCS unit hydrograph transformation. Flow rates are routed via river network using Kinematic Wave. Strengths of the developed model are: (1) very detailed representation of catchment land use and soil cover; (2) relatively fast calculation of the catchment characteristics; (3) rainfall as a main input in the model, no other meteorological observations required. A weakness of the model is that the calculation of effective rainfall is based on statistical, rather than physical description of soil characteristics. Although physical phenomena-based models provide a potentially more correct description of the hydrological processes in the catchment, they also require large amounts of input data and information along with considerable expertise and computation time.

8.2.3.2.3 *Calibration of hydrological model*

Calibration of the Hydrological HEC-HMS model was conducted for a series of a hydrological events from 2011 until 2015. The input rainfall for hydrological model was used from the existing rain gauge located in Dunmanway.

The calibration was obtained for three control points (Long Bridge – HS20008, Bealaboy – HS20016 and Bandon – HS20001) for years 2011 (Figure 11.89), 2012 (Figure 11.90), 2013 (Figure 11.91), 2014 (Figure 11.92) and 2015 (Figure 11.93). Comparison of model results for different catchment conditions, AMCI (dry) and AMCII (wet) for April 2013 is shown in Figure 11.94. The Calibration results are shown in Annex N.

Figure 8.13. Control points (HS2008, HS20016 and HS2001) for calibration of Bandon HEC-HMS model.

8.2.3.2.4 *Model set-up using Hec-GeoHMS*

The HEC-HMS model for Bandon catchment was developed using HEC-GeoHMS software package [192, 193]. Satheeshkumar et al. used similar GIS based approach for the development of SCS-CN model [194].

(a) Terrain processing

The main physical parameters of the model (flow direction, flow accumulation, stream alignment lengths and slopes, subcatchment delineation, catchment centroids and calculation of longest flow paths) requires processing of input raster and vector as shown in Table 8.8. The whole process is explained in a tutorial [195].

Table 8.8. Input and output data used in the GIS processing for development of the HEC-HMS model.

Data type	**Input data**	**Output data**
Raster	**Digital Terrain Model (DTM)** source: **Ordinance Survey Ireland**	Hydro DTM (Digital Elevation Model after reconditioning and filling sinks)
		Flow Direction Grid
		Flow Accumulation Grid
		Stream Grid
		Watershed Slope
		Catchment Grid
Vector	**Catchment polygon** source: **Environmental Protection Agency**	Sub-catchment delineation Polygon
	River Polylines source: **Environmental Protection Agency**	River Channel polylines Centroids Longest Flow-paths

(b) Curve Numbers

The Curve numbers are obtained based on the overlay of the Land Use and soil hydrological groups.

i. Bandon catchment Land Use

Bandon catchment Land use was defined by GIS processing using the Corina Landcover 2012 dataset. This is one of the main inputs for defining of the Curve Numbers for the Bandon catchment.

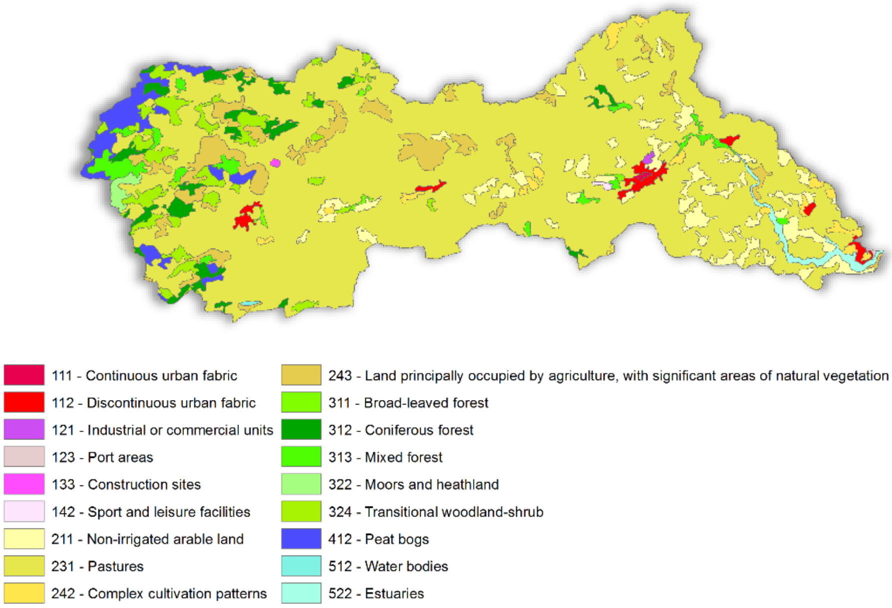


Figure 8.14. *Corina Land Cover for Bandon Catchment.*

Table 8.9. Land use of Bandon catchment.

Level 1	Level 2	ID	Level 3	Area_km2	%
1 Artificial surfaces	11 Urban fabric	111	Continuous urban fabric	0.306582	0.05%
		112	Discontinuous urban fabric	6.280115	1.07%
	12 Industrial, commercial a	121	Industrial or commercial units	0.382053	0.06%
		123	Port areas	0.102000	0.02%
	13 Mine, dump and constru	133	Construction sites	0.269671	0.05%
	14 Artificial, non-agricultur	142	Sport and leisure facilities	0.392045	0.07%
2 Agricultural areas	21 Arable land	211	Non-irrigated arable land	34.314248	5.82%
	23 Pastures	231	Pastures	428.656157	72.73%
	24 Heterogeneous agricult	242	Complex cultivation patterns	8.005694	1.36%
		243	Land principally occupied by agriculture, with signi	36.195556	6.14%
3 Forest and semi na	31 Forests	311	Broad-leaved forest	1.319798	0.22%
		312	Coniferous forest	15.562882	2.64%
		313	Mixed forest	10.935336	1.86%
	32 Scrub and/or herbaceou	322	Moors and heathland	2.333637	0.40%
		324	Transitional woodland-shrub	23.554662	4.00%
4 Wetlands	41 Inland wetlands	412	Peat bogs	15.325954	2.60%
5 Water bodies	51 Inland waters	512	Water bodies	0.258117	0.04%
	52 Marine waters	522	Estuaries	5.164175	0.88%

ii. Hydrological soil groups

The Soil hydrological groups (A-excessive and well drained; B-Moderately drained; C-Imperfectly drained and D-Poorly drained) are defined based on the Irish Soil Classification System (Indicative Soil Drainage Map HSMD2.0) [196].

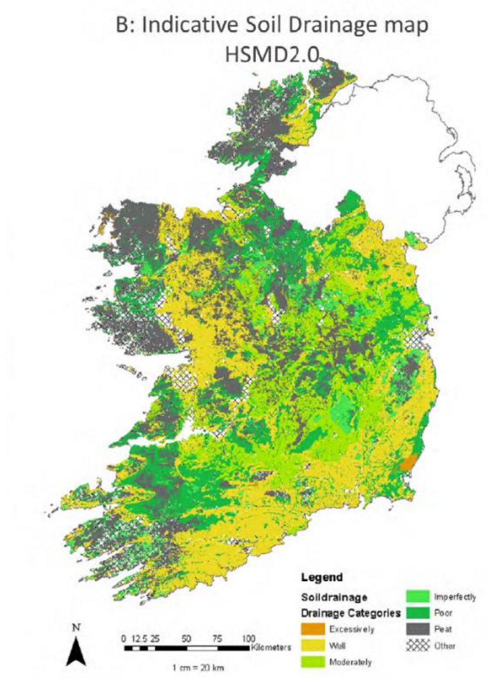


Figure 8.15. Indicative Soil Drainage map HSMD2.0 for Ireland.

(c) Calculation of Curve Numbers

The Curve number CN(II) [191] is calculated by overlaying of the Corine Land Cover (CLC 2012) map with Soil Hydrological group map for the medium Antecedent Moisture Conditions (AMC II). The calculated Curve Number CN(II) was transformed into the Curve number for dry and saturated soil conditions, CN(I) (eqn 8.10) and AMCIII (eqn 8.11) respectively, see Figure 8.16 below.

$$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)} \qquad \text{(eqn 8.10)}$$

$$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)} \qquad \text{(eqn 8.11)}$$

The lookup tables for the calculation of Curve Number for dry (AMCI), medium (AMCII) and saturated (AMCIII) catchment conditions based on the Corine Land Cover (CLC 2012) map and Soil Hydrological group map is shown in Table 8.10. Curve Number Estimation for a Small Urban Catchment was conducted by Banasik et al. [197]. Akbari [198] introduces a slope adjustment for the CN numbers.

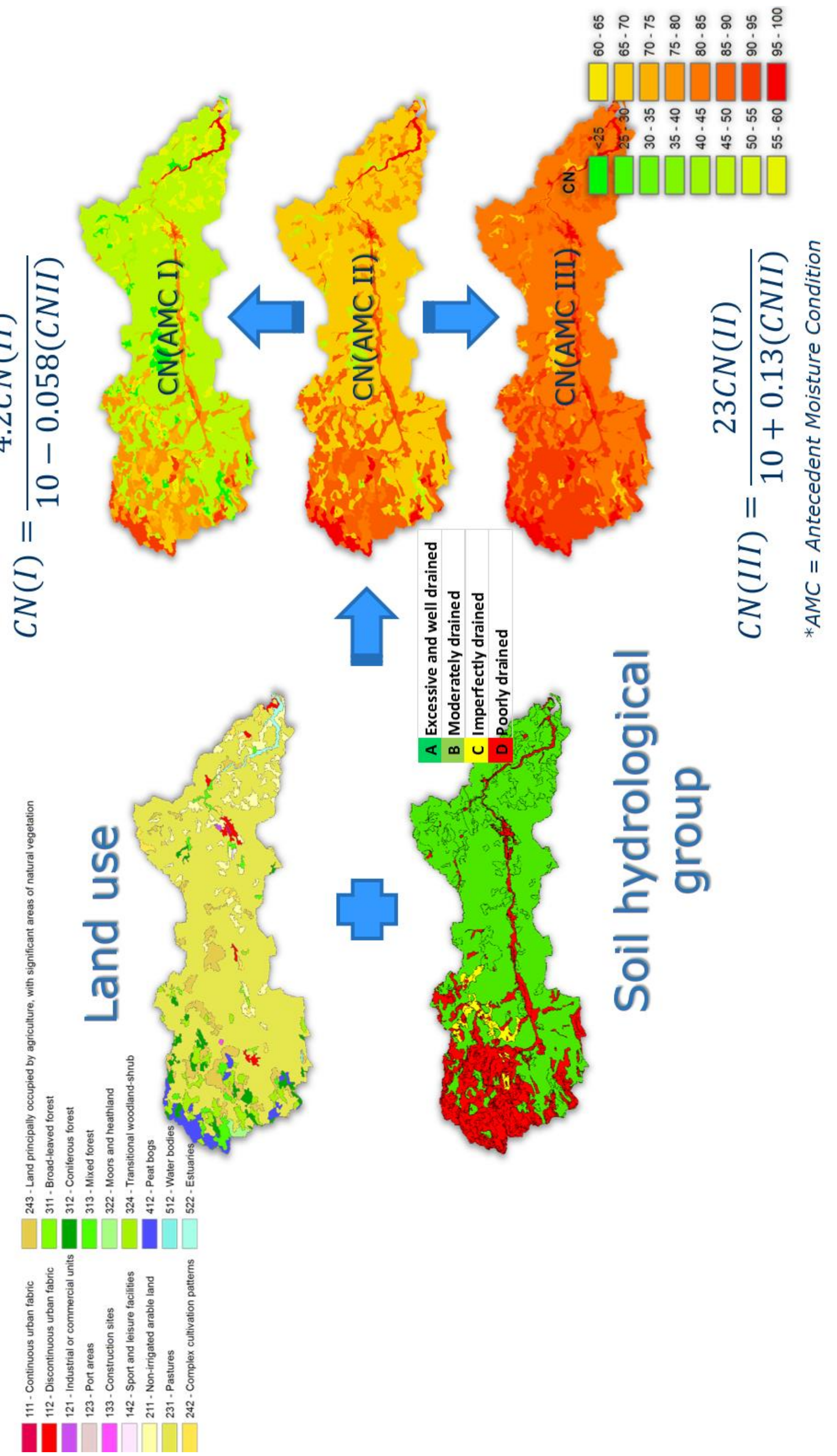


Figure 8.16. Calculation of Curve Numbers (AMCI, AMCII and AMCIII) for Bandon Catchment.

Table 8.10. Lookup table for calculation of Curve Number (AMCI, AMCII and AMCIII) based on land cover and Hydrological Soil group.

a) Curve Number for dry conditions (AMCI)

Code	Corine Land Cover Group	Soil A	Soil B	Soil C	Soil D
111	Continuous urban fabric	95	95	95	95
112	Discontinuous urban fabric	58	70	79	83
121	Industrial or commercial units	64	75	81	85
123	Port areas	95	95	95	95
133	Construction sites	58	72	81	87
142	Sport and leisure facilities	30	47	60	69
211	Non-irrigated arable land	58	72	81	87
231	Pastures	47	61	72	77
242	Complex cultivation patterns	48	60	67	74
243	Land principally occupied by agriculture, with	28	46	58	67
311	Broad-leaved forest	42	45	54	61
312	Coniferous forest	42	56	70	77
313	Mixed forest	42	46	63	70
322	Moors and heathland	28	46	58	75
324	Transitional woodland-shrub	28	46	58	75
412	Peat bogs	95	95	95	95
512	Water bodies	100	100	100	100
522	Estuaries	100	100	100	100

b) Curve Number for medium conditions (AMCII)

Code	Description	Soil A	Soil B	Soil C	Soil D
111	Continuous urban fabric	98	98	98	98
112	Discontinuous urban fabric	77	85	90	92
121	Industrial or commercial units	81	88	91	93
123	Port areas	98	98	98	98
133	Construction sites	77	86	91	94
142	Sport and leisure facilities	51	68	78	84
211	Non-irrigated arable land	77	86	91	94
231	Pastures	68	79	86	89
242	Complex cultivation patterns	69	78	83	87
243	Land principally occupied by agriculture, with	48	67	77	83
311	Broad-leaved forest	63	66	74	79
312	Coniferous forest	63	75	85	89
313	Mixed forest	63	67	80	85
322	Moors and heathland	48	67	77	88
324	Transitional woodland-shrub	48	67	77	88
412	Peat bogs	98	98	98	98
512	Water bodies	100	100	100	100
522	Estuaries	100	100	100	100

c) Curve Number for saturated (wet) conditions (AMCIII)

Code	Description	Soil A	Soil B	Soil C	Soil D
111	Continuous urban fabric	99	99	99	99
112	Discontinuous urban fabric	89	93	95	96
121	Industrial or commercial units	91	94	96	97
123	Port areas	99	99	99	99
133	Construction sites	89	93	96	97
142	Sport and leisure facilities	71	83	89	92
211	Non-irrigated arable land	89	93	96	97
231	Pastures	83	90	93	95
242	Complex cultivation patterns	84	89	92	94
243	Land principally occupied by agriculture, with	68	82	89	92
311	Broad-leaved forest	80	82	87	90
312	Coniferous forest	80	87	93	95
313	Mixed forest	80	82	90	93
322	Moors and heathland	68	82	89	94
324	Transitional woodland-shrub	68	82	89	94
412	Peat bogs	99	99	99	99
512	Water bodies	100	100	100	100
522	Estuaries	100	100	100	100

8.2.4 FFS main output: Official forecast

The Selection of an official forecast is done using API index. The hydrological model HEC-HMS runs for three Antecedent Moisture Conditions (AMC I, II and III) for dry, medium and saturated soil conditions. When the wetness of the catchment (dry, medium or saturated soil condition) is known, an official forecast for model AMC I, II or III can be selected.

In order to do so, an Antecedent Precipitation Index (API index) [199, 200] is utilised. The API index is described by the equation below:

$$API_d = P_d + kP_{d-1} + k^2 P_{d-2} + \ldots + k^n P_{d-n} \qquad \text{(eqn 8.12)}$$

where,

API_d - is the Antecedent Precipitation Index for day d [mm]

k - is an empirical decay factor less than one (0.85-0.95) [1]

y_0 - is rainfall for day d [mm]

Essentially, the API index is a running day by day measure of catchment wetness based on the rainfall that has occurred over preceding days. The more recent rain has higher impact on the value of API index than rain from previous days. Ladson [201] gave a detailed explanation of API index.

The Bandon FFS currently calculates the value of the API index for the last 5 days. It uses an empirical decay factor of k=0.90.

The system would select AMC I model if the API index is less or equal than 20mm. The official forecast would be issued using the AMC II model in case that API index is between values 20 and 50. For API index higher than 50, the official forecast takes the AMC III model results.

8.3 Practical application of FFS in BMS – novelty and adaptation to extreme flood events

As stated in section 8.1, the Bridge Management System could benefit the most from Local FEWS as the Local FEWS are capable producing a site-specific warning(s). FEWS in Bridge Management Systems can be applied for centralised monitoring and viewing of real time data; as a warning and planning tool during works in the river (it has a potential to become prerequisite for implementation of flood relief schemes – scale of works in the river of a month or longer); as tool for planning of bridge inspections up to 10 days in advance (low or high water levels); as a source of information relevant to make decisions on bridge closure. Bridge closures are rare, but with an increase in information accessibility from FEWS, this practice could change.

As a demonstration, a new FFS on Bandon Catchment was developed. The rationale for the development of Bandon catchment was availability of data from pre-existing Bandon FEWS and the possibility for the comparison between two different systems (one based on correlation of water levels from upstream gauge and the second which is based on the meteorological forecast and hydrological model).

8.3.1 Scheduling of bridge inspections up to 14 days in advance

Bandon FFS is a system that provides flood forecasts up to 14 days in advance. This is possible due to input rainfall dataset (see section 8.2.2.4). Typically, bridge scour inspection is conducted during the low water level condition. This is especially useful for smaller bridges where walking around the bridges is possible.

With the support of a flood forecasting system, the supervisor managing the bridge inspections does not need to rely on weather forecasts only. The Flood forecasting system that can predict flood up to 14 days in advance, can also predict low flows 14 days in advance.

Water level at the bridge is much more useful information than predicted rainfall to an on-site engineer. It is estimated that bridge inspections are planned on a weekly basis. Site engineers and their supervisors will be able to rely on the FFS to get a realistic sense of which hydrological conditions to anticipate at the bridge up to 14 days in advance.

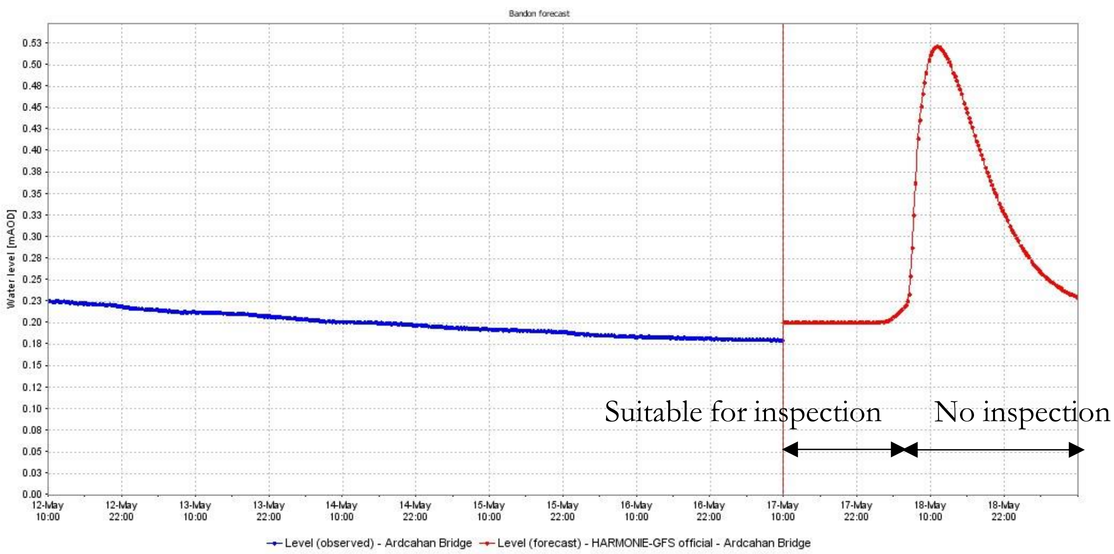


Figure 8.17. Scheduling of inspection.

8.3.2 DSS1 - Scheduling of bridge inspections based on observed flood levels

A simple decision support system that takes into account scour condition of the bridge (ScCR) and the flood level(s) at the bridge is introduced here. The data operator system algorithm goes through the rules shown in Table 8.12 after each simulation. The lookup algorithm issues the recommended action as shown in below section. This system is useful in order to improve scour inspection intervals. This system is also useful for bridges with ScCR of 3 or above. The time to next inspection, unless it is not binding with National Code, can be either reduced or increased and the inspections could be done during or post flood events. A similar approach is accepted in Austrian code "RVS 13.03.11" which allows bridge inspections with greater intervals in the case where the bridge has monitoring sensors in place. In the next section a framework for automation of recommended actions based on flood levels and bridge scour conditions will be outlined.

8.3.2.1 Flood levels

It is assumed that each bridge either has a water level gauge installed at the bridge or that a water level gauge is located within 1km upstream or downstream of the bridge. For each water level gauge flood level zones are defined. The list below gives an approximate indication on how to easily define flood zones.

Table 8.11. Flood levels at the bridge with corresponding flow rate return period (RP).

No	Flood Level	Description
1	Green RP: 1 year	There is no flood.
2	Yellow (Low) RP: 2 years	water level / flow rate at which 50%-75% of the free flow area through the bridge is submerged
3	Yellow (High) RP: 10 years	meaning water level or flow rate at which 75%-90% of the free flow area through the bridge is submerged
4	Orange RP: 25 years	meaning water level or flow rate at which >90% of the free flow area through the bridge is submerged
5	Red RP: 100 years	meaning water level or flow rate at which bridge opening is fully submerged and there is a danger of pressure flow through the bridge or overtopping of the bridge

8.3.2.2 Actions and recommendations

Based on a combinations between flood level and bridge condition rating, automated DSS issues the following actions:

A1. No Action Required

A2. Alert personnel nearby. Visual inspection required after flood event is over.

A3. Full L1/L2 scour inspection required after flood event.

A4. Engineer on-site during the flood event required for observation

A5. Close the bridge.

The lookup matrix showing which combination of Scour Condition Rating (ScCR) and flood levels triggers which action:

Table 8.12. DSS Actions based on recorded Flood levels and bridge Scour Condition Rating.

		Flood Levels				
		Green	**Yellow (Low)**	**Yellow (Hi)**	**Orange**	**Red**
Scour Condition Rating (ScCR)	**0**	A1	A1	A1	A1	A1
	1	A1	A1	A1	A2	A2
	2	A1	A1	A2	A2	A3
	3	A1	A2	A2	A3	A4
	4	A1	A2	A3	A4	A5
	5	A5	A5	A5	A5	A5

8.3.2.3 Results of application of DSS based on Flood levels

The rules defined in section above were applied on 101 railway bridges in Ireland (Data block 2 from Chapter 7). The results (Figure 8.18) on 101 bridges showed how the actions recommended for each are dynamic and relative to flood levels.

For the case "Green flood level", e.g. no flood, it can be seen that no action is required for the whole range of ScCR from 0-4. The bridge inspections are scheduled according to the recommended years to next inspection based on the last inspection. Bridges that were closed remain closed even when there is no flood event. In our case one bridge is closed. The annual exceedance probability of this scenario to occur is 100%.

For the case "Yellow (low) flood level" it can be seen that 67 bridges still require no action, but 33 bridges require visual inspection immediately after the flood. Already closed bridges remain closed. The annual exceedance probability of this scenario to occur is 50%.

For the case "Yellow (high) flood level" only 18 bridges do not require any action and 78 bridges require visual inspection. Four (4) bridges require L1 or L2 bridge inspection, depending on the complexity of the bridge (Level 1 or Level 2 bridge). Already closed bridges remain closed. The Annual exceedance probability of this scenario to occur is 4%.

For the case "Orange flood level" only 10 bridges do not require any action and 57 bridges require visual inspection and 29 bridges require L1 or L2 bridge inspection. Already closed bridges remain closed.

For the extreme case "Red" only 10 bridges do not require any action and 8 bridges require visual inspection and 49 bridges require L1 or L2 bridge inspection. Already closed bridges remain closed. Total 5 bridges are now recommended to be closed immediately. Annual exceedance probability of this scenario to occur is 1%.

System demonstrates how the resources can be minimised during hydrological events of more frequent occurrence (≥50% AEP)

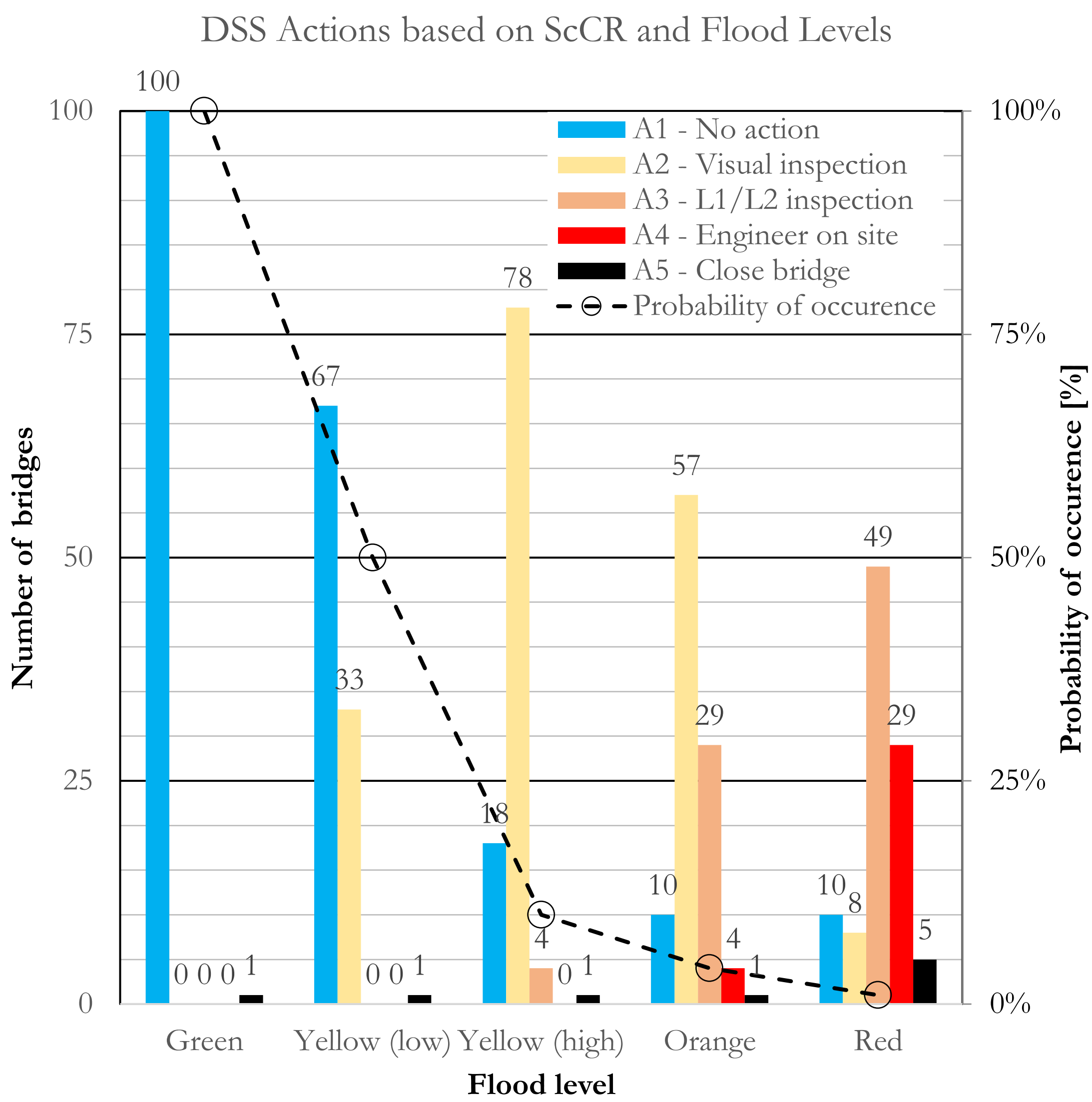


Figure 8.18. Results of applying DSS1 on 101 bridges for a range of different flood levels.

8.3.3 DSS2 - Scheduling bridge inspections based on observed rainfall (Crisis management)

During an extreme flood event management of the bridge could become chaotic in case that the system is not set-up properly, e.g. if necessary information is not available readily available. Anderson et.al. [202] study that suggests the strong connection between the rainfall distribution and the extent of damage to bridges is a good example of how FFS can enhance BMS. A simple overlay of rainfall distribution with bridge locations (Annex O), as developed in Anderson et. al.'s study [202] is a very good example of type of information that could be used for a bridge manager decision on where to deploy on-site personnel during the rainfall event.

Following similar principles, DSS2 system logic is developed as part of this thesis. In order to reduce the time for the analysis to a minimum, a set of rules is defined in order to manage on-site personnel in the most efficient way, without delay. The bridge manager does nothing to analyse flows or rainfall, as the system does this automatically and issues notifications with a list of the bridges that need to be inspected or closed immediately. In this way, the personnel respond to the flood immediately and focus their inspection only on bridges that require their attention.

This system is cheaper than the DSS1 system described in section 8.3.2 as it requires lower number of sensors (rain gauges or water level gauges) and relies on the existing rainfall forecasting products.

The workflow from observation to action is shown in Figure 8.19. The process is triggered in the case that a rain gauge records rain intensity according to desired specifications (indicative intensity is given in Figure 8.19). Alternatively, the process can be triggered by observed water levels (water level of yellow (low) or higher). As the study area is located in Ireland, we use the HARMONIE rasterised dataset in order to obtain the rain distribution over the catchment. Since the first release of Bandon FFS in 2017, it was noted that the latest HARMONIE rainfall forecast has a very satisfactory accuracy in timing and amount of rainfall when compared with rain gauge stations.

The Delft FEWS data operator has functionality to transform and average a rasterised dataset into predefined polygons, that, in our case represents the Bandon sub catchments. In this way it is possible to create list of active polygons for which it is possible to indicate active sub catchments. This is a very important feature in order to improve the forecast, e.g. reduce number of rainfall alerts and to narrow the area required for the inspection.

The sub-catchments that have rainfall intensity above designated thresholds (for initial set-up it is recommended to check the system for rainfall events with total rainfall of 50mm in the last 24 hours and 25mm of rainfall in the last 1 hour) are highlighted. Once the sub-catchment polygons are highlighted, the system lists all the bridges located within the highlighted polygons. Further operations can be done either within the FFS or on separate server with integrated Decision Support System (DSS).

For the selected list of the bridges, DSS checks Scour Condition Ratings (ScCR) of the bridge. Based on the ScCR, the DSS recommends actions that are defined in Table 8.13 below.

Table 8.13. DSS2 Actions based on recorded rainfall – useful for Crisis management.

<table>
<tr><td rowspan="6">Scour Condition Rating (ScCR)</td><td>0</td><td rowspan="3">1</td><td rowspan="3">No Action</td></tr>
<tr><td>1</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td><td>3</td><td>L1/L2 Inspection after the flood required</td></tr>
<tr><td>4</td><td>4</td><td>Engineer on site during the flood. Conduct L2 inspection after the flood.</td></tr>
<tr><td>5</td><td>5</td><td>Close the bridge</td></tr>
</table>

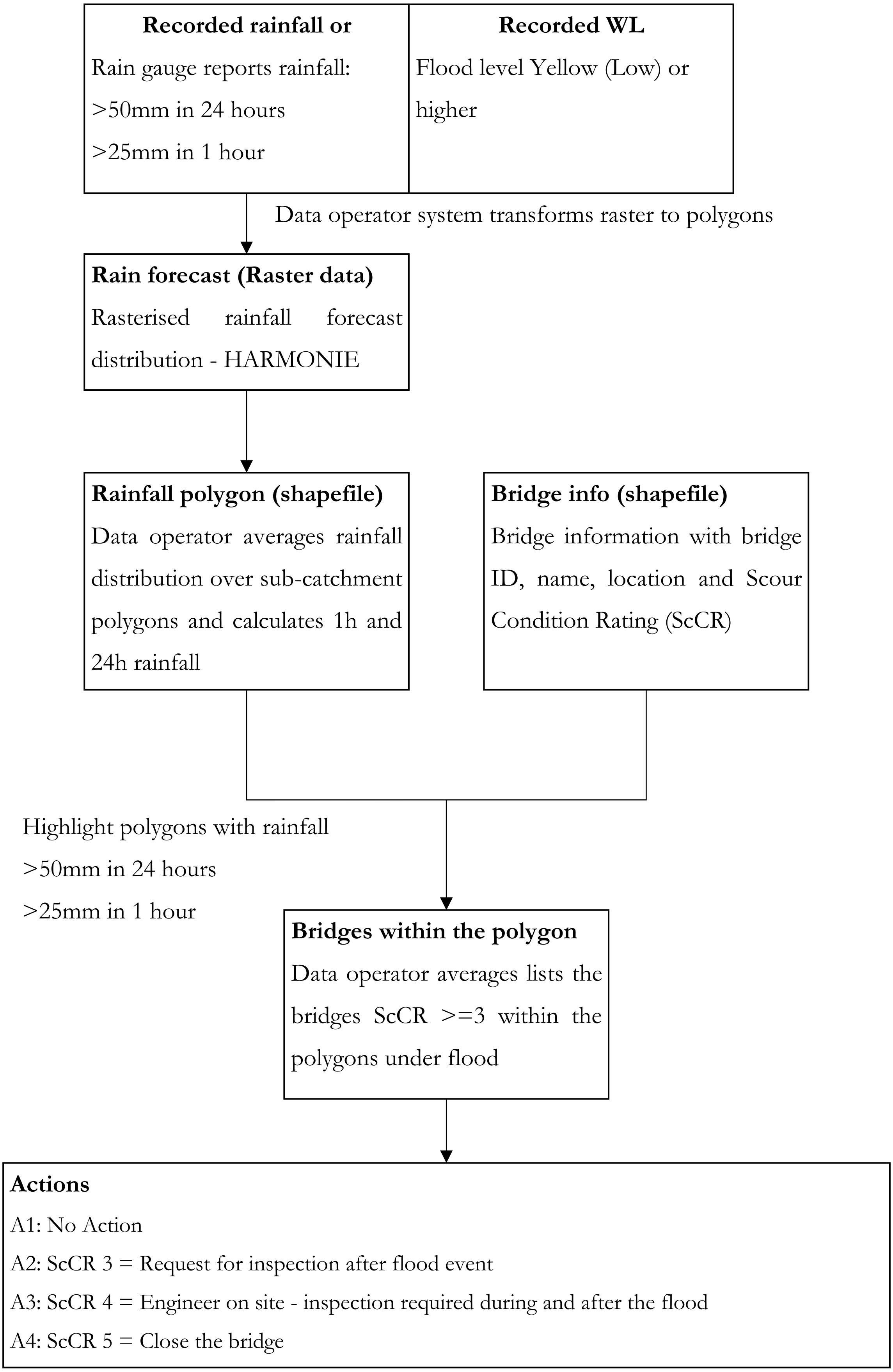


Figure 8.19. Workflow for DSS based on recorded rainfall

8.3.3.1 Application of DSS based on rainfall observations

It should be noted that the probability of occurrence of this type of events is expected to have return period of 25-years or more (4% AEP or lower). Due to the lower frequency of occurrence it would be advisable to establish protocols for the simulation of such events and responses as continuous on-site training and exercises during these simulations would ensure readiness of staff and engineers.

The results of a simulation of an extreme rainfall event show that significant savings resources could be made if the DSS2 is available to bridge owners. Instead of inspecting 101 bridges, the personnel would be focused on inspections of 34 bridges, meaning reducing the need for personnel engagement by 66.33%.

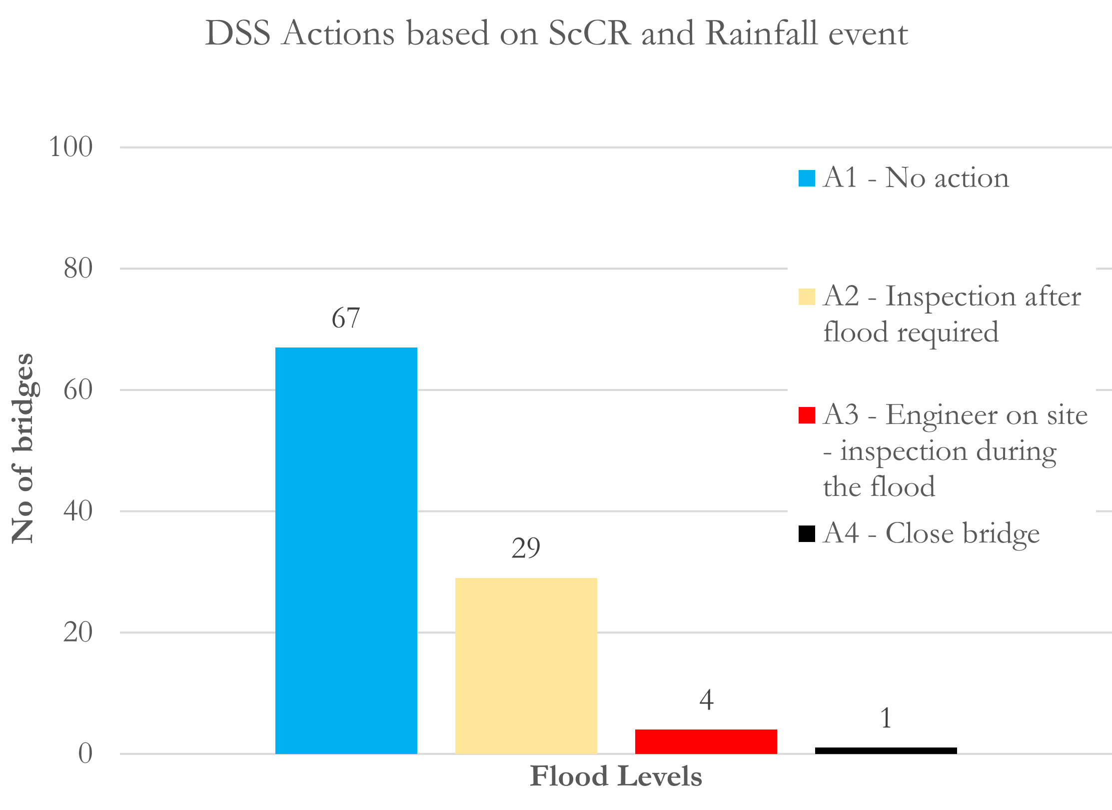


Figure 8.20. Results of applying DSS2 on 101 bridge in Ireland.

8.3.4 Scour Depth Model (SDM)

8.3.4.1 Method

A proposed approach for the calculation of theoretical scour depths is described in Figure 8.21 and it is similar to Park et al.'s [85] approach. In the first step a) the system reads the real water levels Y [m] for the time increment t_n. In the second step b), flow velocity v [m/s] for time increment t_n is obtained from the correlation Y-v of water levels Y [m] and flow velocities v [m/s]. Correlation Y-v is obtained from the one-, two- or three-dimensional hydraulic models simulations. In the third step c) theoretical scour depth D_s [m] is calculated using one of empirical formulas for scour depth around bridge piers / abutments [76-84]. Depending on the formula used, scour depth D_s [m] is calculated based on the following parameters: Flow velocity v [m/s], shear stress τ [N/m^2], Froude number Fr [1], water depth Y [m], median grain size $D50$ [mm] and bridge geometry [85, 86].

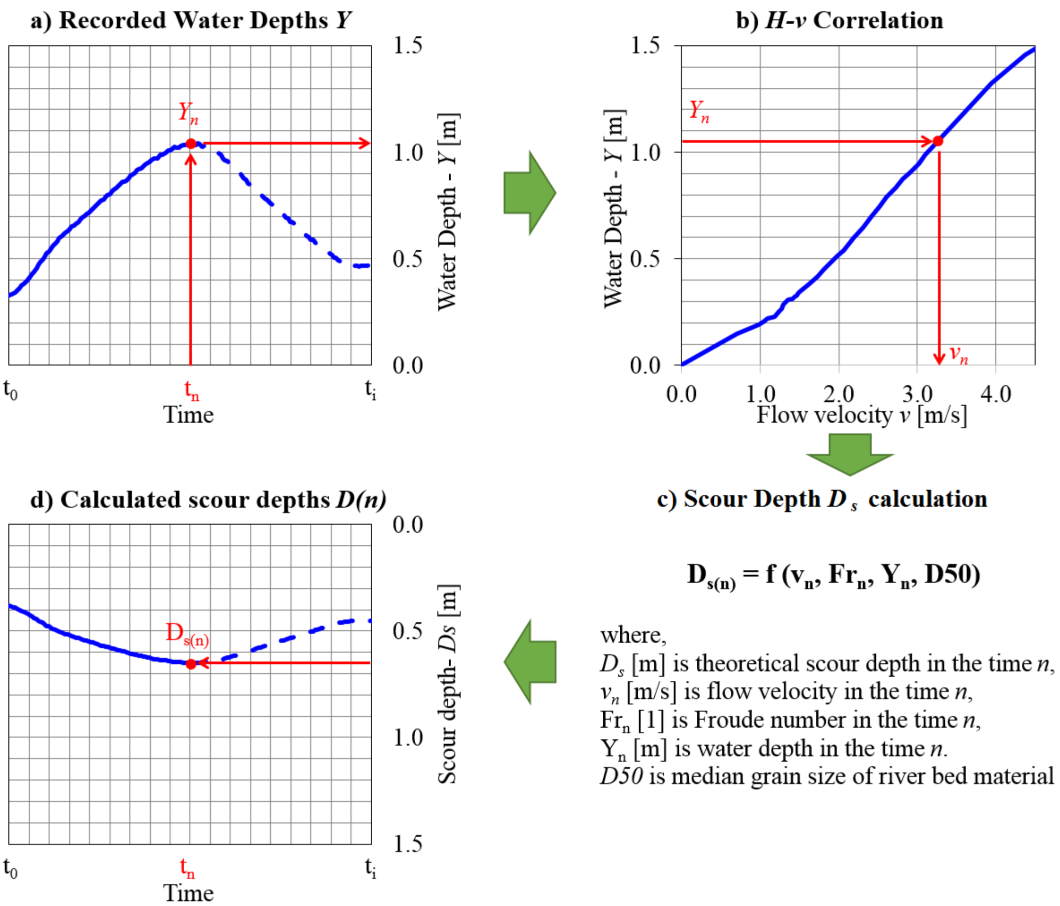


Figure 8.21 General methodology for scour prediction from water levels measurements.

8.3.4.2 Governing Equations for scour depth calculation

The depth of scour at the bridge can be estimated on the basis of the results of the water depth and the flow velocities at the bridge profile. The hydraulic parameters are obtained from the physical or mathematical hydraulic model. The depth of scour is firstly estimated for the constriction scour and the local scour at the bridge piers and abutments. The results of the two calculations are added together to give the total scour depth. As the general scour is not quantified in the estimate of the total scour depth, this significant element of bridge stability will be comprised in the risk assessment and final conclusions.

Calculation of scour depth at bridges in the HEC-RAS model is based on the methods outlined in the HEC-18 [84] and CIRIA [4] documents. The scour calculations are supplementary to the 1D steady flow calculations. In the HEC-RAS model, the scour estimation at the bridge includes calculation of constriction scour, local scour at bridge piers and abutments, and total scour at bridge piers and abutments.

8.3.4.2.1 *Constriction scour*

In general, constriction scour is deduced from (eqn 8.13):

$$y_S = y_2 - y_0 \qquad \text{(eqn 8.13)}$$

where,

y_s - average depth of constriction scour [m]

y_2 - average depth after scour in contracted section [m]

y_0 - average depth in main channel/floodplain at contracted section before scour [m]

In order to calculate constriction scour, the HEC-RAS model needs to determine if the flow upstream is transporting bed material (live-bed). The program calculates the critical velocity v_c for the beginning of motion (for the D_{50} size of bed material) and compares it to the mean velocity V of the flow in the main channel or overbank area upstream of the bridge at the approach section. If the critical velocity v_c is greater than the mean velocity at the approach section ($v_c > V$), then clear-water scour is assumed. If the critical velocity

v_c is less than the mean velocity at the approach section ($v_c < V$), then live-bed scour is assumed.

Constriction scour in HEC-RAS can be computed using Laursen's clear water or Laursen's live bed equations.

Clear water constriction scour

$$y_2 = y_1 \left[\frac{Q_2^2}{C D_m^{2/3} W_2^2} \right]^{3/7} \quad \text{(eqn 8.14)}$$

where,

D_m - diameter of smallest non-transportable particle in the bed material in the contracted section [m]

D_{50} - median diameter of the bed material [m]

C - coefficient with value of 40.0 for metric

Live bed constriction scour

$$y_2 = y_1 \left[\frac{Q_2}{Q_1} \right]^{6/7} \left[\frac{W_1}{W_2} \right]^{K_1} \quad \text{(eqn 8.15)}$$

where,

y_2 - average depth after scour in contracted section [m]

y_1 - average depth in main channel/floodplain at approach section [m]

Q_1 - flow in the main channel/floodplain at the approach section, which is transporting sediment [m^3/s]

Q_2 - flow in the main channel/floodplain at the contracted section, which is transporting sediment [m^3/s]

W_1 - bottom width in the main channel/floodplain at the approach section [m]

W_2 - bottom width in the main channel/floodplain at the contracted section less pier widths [m]

All of the variables, except an exponent *K1* [1] for live bed constriction scour and mean diameter of bed material D_{50} [mm] are obtained automatically from the HEC-RAS output file.

8.3.4.2.2 *Local scour at bridge piers*

Pier scour occurs due to the acceleration of flow around the pier and the formation of flow vortices (e.g. horseshoe vortex). The horseshoe vortex removes material from the base of the pier, creating a scour hole. As the depth of scour increases, the horseshoe vortex decreases until equilibrium is reached and the scour hole stops expanding. The factors that affect the depth of local scour are: flow velocity just upstream of the bridge, depth of flow, width of the pier, length of the pier if skewed to the flow, size and gradation of bed material, skew angle, shape of pier (nose), bed configuration and formation of debris or ice. Pier scour can be computed by either the Colorado State University (CSU) or Froelich equation.

CSU local scour at pier

$$y_S = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot a^{0.65} \cdot y_1^{0.35} \cdot Fr_1^{0.43} \qquad \text{(eqn 8.16)}$$

where,

y_s - depth of scour [m]

K_1 - correction factor for pier nose shape

K_2 - correction factor for angle of attack of flow

K_3 - correction factor for bed condition

K_3 - correction factor for armouring of bed material

y_1 - flow depth just upstream of bridge pier [m]

Fr_1 - Froude number just upstream of pier.

Froehlich local scour at pier

$$y_S = 0.32 \cdot \phi \cdot (a')^{0.62} \cdot y_1^{0.47} \cdot Fr_1^{0.22} \cdot D_{50}^{-0.09} + a \qquad \text{(eqn 8.17)}$$

where,

f - correction factor for pier nose shape (=1.3 for square nose piers; =1.0 for rounded nose piers; =0.7 for sharp nose (triangular) piers).

a - pier width [m]

a' - projected pier width [m]

The user is only required to enter pier nose shape *K1* (1), the skew angle to the bridge piers (°), the condition of the bed *K3* (1) and D_{95} size of the bed material (mm). All other values are automatically obtained from the HEC-RAS output file.

8.3.4.2.3 *Local scour at bridge abutments*

Local scour at abutments occurs when abutments obstruct the flow. The obstruction of flow forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment; where it forms a vertical wake vortex at the downstream end of the abutment. Abutment scour data can be computed by either the HIRE or Froelich equation.

HIRE equation for local scour at abutment

$$y_S = 4.0 \cdot y_1 \cdot \left(\frac{K_1}{0.55}\right) \cdot K_2 \cdot Fr_1^{0.33} \qquad \text{(eqn 8.18)}$$

where,

- y_s - depth of scour [m]
- y_1 - depth of flow at the toe of abutment on the overbank or in the main channel [m]
- K_1 - correction factor for abutment shape
- K_2 - correction factor for angle of attack
- Fr_1 - Froude number based on flow velocity and depth upstream of the abutment toe

Froehlich's equation for local scour at abutment

$$y_S = 2.27 \cdot K_1 \cdot K_2 \cdot (L')^{0.43} \cdot y_a^{0.57} \cdot Fr^{0.61} + y_a \qquad \text{(eqn 8.19)}$$

where,

- y_s - depth of scour [m]
- K_1 - correction factor for abutment shape
- K_2 - correction factor for angle of attack
- L' - length of abutment (embankment projected normal to the flow [m])
- y_a - average depth of flow on the floodplain at the approach section [m]
- Fr_1 - Froude number based on flow velocity and depth upstream of the abutment toe

8.3.4.3 Application of Scour Depth Model on Bandon FFS

Within the Bandon FFS, a Scour Depth Model (SDM) is set-up for an arch bridge Meelon in Ireland, Co. Cork over the Bridewell river (Tributary of Bandon River) and uses empirical equations for pier (eqn 8.16) and abutment (eqn 8.19) scour depth calculations recommended by *CIRIA* [4].

Flow velocities and Correlation *Y-v* for a range of flows are obtained from a 2D hydraulic model [203].

The SDM model results for 28th December 2017 forecast are shown in Figure 8.22.

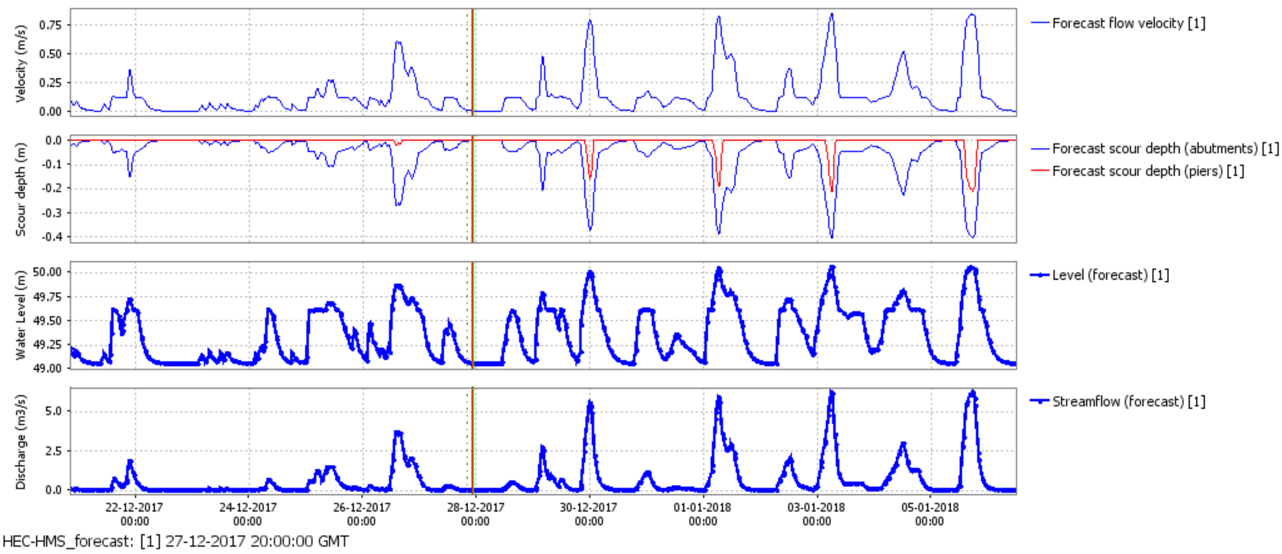


***Figure 8.22** Scour Depth Model(s) for Meelon Bridge (Bridewell River) on 28th Dec 2017.*

Only periodic monitoring of river bed changes at Meelon bridge is obtained. The Digital Terrain Model (DTM) from one of the periodic surveys is shown in Figure 8.23. This information is useful and cost-effective, but the preferred system requires measurements of scour depth on the daily or even hourly interval in order to test developed Scour Depth Model.

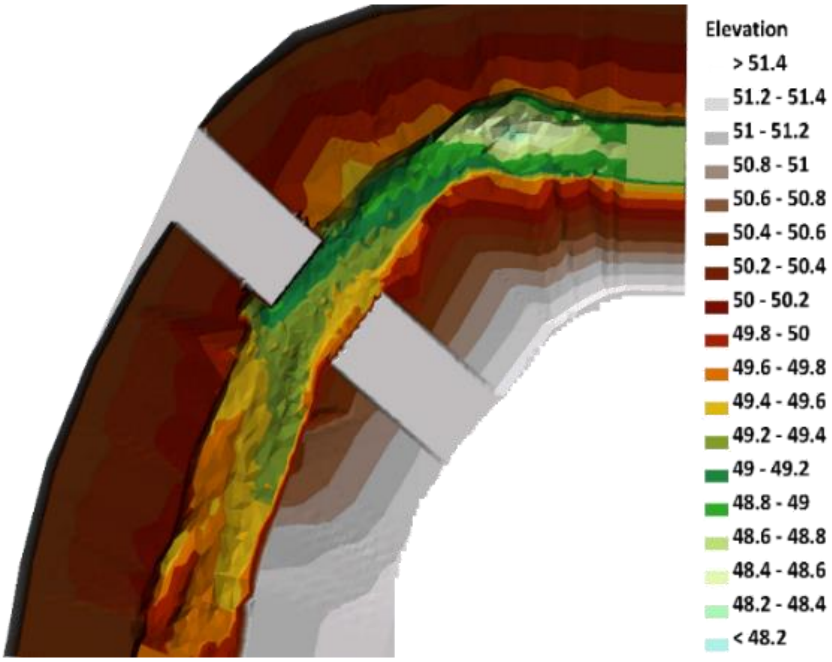


***Figure 8.23** Digital Terrain Model for Meelon (Bridewell river) from periodic survey [203].*

8.3.4.4 Conclusions on scour model

The Scour Depth Model (SDM), although robust, makes the Bandon FFS unique (the author is not aware of previous implementation of any other scour models in Flood Warning systems). The SDM needs be set for each bridge as the it requires specific information on bridge geometry, rating curves, flow velocity ratings and river bed material. Water levels and flow rates are correlated to the flow velocity which provide the basis for the prediction of scour depth model.

The current lack of described method is that empirical formulas to predict scour depth very often overestimate of theoretical scour when compared to actual scour [143-145]. In another study, Park et al. [85] calculated scour depths using four equations [51, 76, 77, 80] and the results showed smaller than average scour depths. However, this underestimation of scour depth equations is attributed to the depth of rock. Mahjoobi et al. study [86] showed that model and regression trees are more efficient than the empirical formulas to predict scour depth. Real-time data of scour depth variations would be valuable information to assess, validate and improve the existing scour depth empirical models. Further on-site data gathering (water levels and scour depth monitoring) can contribute in improvement of scour empirical equations.

Chapter 9
Costs and Benefits of new Inspection and Flood Forecasting Modules

An assessment of the costs of bridge inspections extended with a basic and running costs of prediction module is carried out in this chapter.

In summary, the estimated cost of the new scour module based, bridge inspection(s) is c. €145.00 per bridge for a Level 1 inspection and c. €520.00 for a Level 2 inspection for the case that the inspections are carried out in-house.

The total cost of a prediction module developed for Bandon Flood Forecasting system with a design duration of 2 years is estimated to be c. €67,000.00. A tool called "PREDICT" for a rough assessment of the cost of prediction module is developed. Tool is available on the CD as part of this thesis (see "CostCalculation-PREDICT.xlsx").

9.1 Benefits of Proposed Scour Inspection Module

The benefits of the proposed Scour Inspection Module are summarised below:

- Expand the existing bridge inspections with more focus on scour issues
- Standardisation of the scour inspection methodology
- Reducing the cost of inspection
- Breaking judgement in more components (spread risk of error)
- Split the bridges into categories - simple and complex structures)
- Prioritisation of bridges
- Development of DSS based on flood forecasts for bridge scour inspections, enabling smart scheduling of bridge inspection
- Reporting done automatically by introducing tablet as a tool for bridge inspections and web-based solution

A newly developed Scour Inspection Module, e.g. bridge scour inspection L1 and L2 methodology enables the bridge management to conduct the majority of the inspections with their own personnel. This is achieved by categorising bridges as Level 1 bridges, e.g. simple / single span structures; and Level 2, e.g. complex bridges. Complexity of the bridge is defined by the number of openings and other hydraulic factors, as defined in Table 6.8. Level 1 bridges would often represent the majority of the available bridge portfolio. For example Cork County Council has a portfolio of around 1,400 bridges [6], of which more than 50% would be classified as simple, single span bridges. Level 2 bridges require more expertise and training in hydrology, hydraulics, river morpho-dynamics, soil and structure materials. This suggests that only the part, not the whole of the bridge portfolio might needs to be subcontracted. Assuming that conducting of a single bridge inspections in-house is 30% cheaper than outsourcing the same inspection, The proposed L1 methodology, in a Cork case [6] could accumulate around 15% of savings in overall annual budget for bridge inspections.

Furthermore, both L1 and L2 inspections should lower the time of inspection, e.g. overall cost inspection, whilst increasing safety of the bridge when compared to the existing approach.

The main contribution to lower costs of inspections is standardisation of the bridge inspection and introducing of the ICT technologies which enabled that the time required for reporting is reduced almost to zero.

Bridge inspections for structures over watercourses are now more reliable as there is more focus on scour. Subjectivity of inspector [104] is minimised as the judgement is broken into more components and the final scour condition rating ScCR of the bridge is automatically calculated. Bridges can be now be prioritised based on ScCR and maintenance and decision for repair(s) can be conducted from the most critical bridges to the ones that do not require immediate attention.

Introducing of the newest ICT technologies and communication channels will standardise the methodology approach even more, ensure there is no corporate memory loss and that the bridge inspector has all knowledge about the bridge in one place, readily available. The new methodology check-list approach will ensure no component is overlooked, it will reduce the time required for the inspection, therefore reduce the cost of inspection. Lowering of the costs of the inspection is analysed in the section 9.2 below.

9.2 Cost of Scour Inspection Module

In this section, costs of the bridge inspections for Bekić-McKeogh (Method B1 and B2) and new inspection module (L1 and L2) are analysed. For the new Level 2 bridge inspections, two options were analysed. First option (L2.1) incorporates simplified survey of the river bed that is incorporated in the bridge inspection process and the second option (L2.2) assumes that the survey of the river bed is outsourced and is not part of inspection. The analysed variants with assumed number of inspections per day and required personnel per inspection and report is shown in Table 9.1.

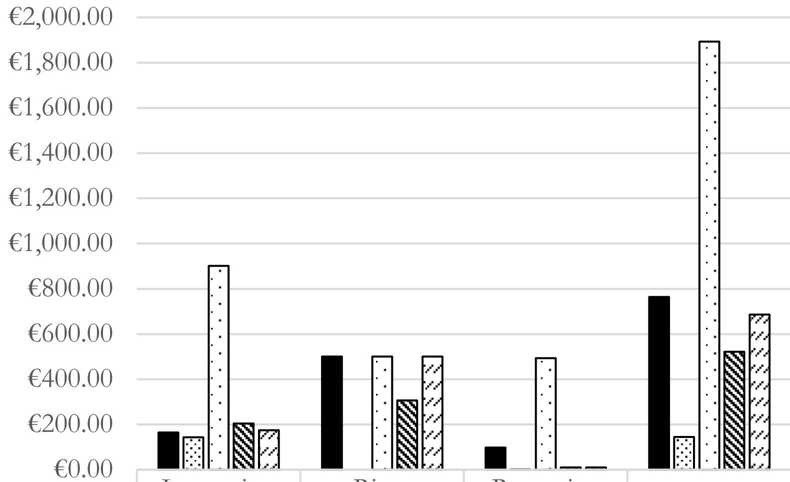
Table 9.1 Required personnel and number of bridge inspections per day

	Bridges inspected per day [1]	*Personnel in inspection [1]*	*Personnel in reporting [1]*
(B1) Bekić-McKeogh Stage 1	5	2	5
(B2) Bekić-McKeogh Stage 2	1	3	5
(L1) New Level 1 inspection	5	2	2
(L2.1) New Level 2 inspection (survey part of inspection)	2	3	2
(L2.2) New Level 2 inspection (survey outsourced)	5	3	2

The calculation includes time required to drive to the bridge, time required to inspect the bridge, time required for reporting, cost of river bed bathymetry survey, travel costs, meals and accommodation. Two levels of Engineers were assumed Technician or Junior Engineer and Senior Engineer. Detailed calculations are shown in Annex P.

The cost of the new bridge inspection module bridge scour inspection(s) is €145.53 per bridge for Level 1 inspection and €521.42 for Level 2 inspection in case that the inspections are obtained in-house. For the comparison, cost of bridge inspection using EIRSPAN[13] system us circa €500.00.

[13] Value obtained from interviewing the site engineers that were involved in EIRSPAN the bridge inspections



	Inspection cost	River Survey Cost	Reporting cost	Total cost
(B1) Bekic-McKeogh Stage 1	€165.32	€500.00	€98.53	€763.85
(L1) New Level 1 inspection	€143.69	€0.00	€1.83	€145.53
(B2) Bekic-McKeogh Stage 2	€900.84	€500.00	€492.67	€1,893.51
(L2.1) New Level 2 inspection (survey part of inspection)	€203.94	€306.48	€11.00	€521.42
(L2.2) New Level 2 inspection (survey outsourced)	€174.51	€500.00	€11.00	€685.51

Table 9.2 Cost of Bridge Scour per single bridge inspection.

The results clearly show how the new inspection module reduces the cost of the inspection for single span and simple bridges. In order to understand where the reduction in cost is coming from, the total cost of bridge scour inspection is divided into:

- Costs of inspection, including time of inspectors at the bridge and travel costs
- Cost of Survey of the River Bathymetry
- Cost of writing inspection reports

When comparing methods B1 and L1, it can be seen that the highest reduction in cost in L1 comes from lowering cost of reporting and cost of River Bathymetry Survey. In the new inspection module (L1), the overall price per bridge inspection is reduced for 80.95%, from €763.85 for Method B1 to €145.42 for L1 bridge inspection. If compared to method B2, the cost is reduced by 92.32%, from €1893.51 to €145.42.

For more complex bridges, the savings are somewhat lower due to need for bathymetry survey. When compared to cost of inspection from method B1 (€763.85), the reduction of the cost is 31.74% and 10.26%, respectively for method L2.2 (€521.42) where a survey of the river bathymetry is integrated in the inspection and for method L2.2 (€685.51) where the river survey is outsourced. Again, savings are significantly greater when comparing to cost of Method B2 inspection. In this case, the cost is reduced by 72.46% and 63.80% for methods L2.1 and L2.2 respectively.

When new Level 1 inspection time spent for reporting is compared to Method B reporting time, it can be seen that the reporting time is shortened from 23 hours to 0.45 hours or by 98.04%. For Level 2 inspection, the reporting time is lowered by 96.39%, to 0.83 hours.

For the new Level 1 inspection, overall bridge inspection and reporting time is reduced to 2.95 hours, or by 88.87% when comparing to method B1 inspection and 93.72% when comparing to method B2 overall inspection time. For the new level 2 inspection, overall bridge inspection is reduced to 12.83 hours, or by 51.58% and 72.70% when compared to overall bridge inspection time of methods B1 and B2 respectively.

If the survey of the river bed that is necessary for the Level 2 bridge inspection is outsourced, the time of the bridge inspection is reduced further reduced to 3.83 hours or, by 85.55% and 91.85% when compared to overall bridge inspection time of methods B1 and B2 respectively, as shown in Table 9.3. Duration of inspection vary between 0.25-0.75 hours when bathymetry survey is not required. In case that bathymetry survey is required, it typically takes whole working day, or up to 24 man hours assuming that three persons are involved in the river bathymetry survey. By simplifying and integrating of the river bed bathymetry survey within the level 2 bridge inspection procedure, total required man hours per bridge were reduced to 12 hours, e.g. by 50%.

Table 9.3 Inspection duration and man hours required.

	Duration of inspection [hours]	*Man hours per inspection [hours]*	*Man hours per report [hours]*	*Total Man Hours [hours]*
(B1) Bekić-McKeogh Stage 1	0.75	3.50	23.00	26.50
(B2) Bekić-McKeogh Stage 2	7.00	24.00	23.00	47.00
(L1) New Level 1 inspection	0.25	2.50	0.45	2.95
(L2.1) New Level 2 inspection (survey part of inspection)	7.00	12.00	0.83	12.83
(L2.2) New Level 2 inspection (survey outsourced)	0.50	3.00	0.83	3.83

9.3 Benefits and effectiveness of Flood Forecast

The benefit of a flood forecasting and warning system can be greatly increased if it is linked to options at the study area scale such as a public awareness campaign and individual property flood protection [204]. Figure 9.1 demonstrates how Early Warning systems have one of the highest benefit to Cost ratio and the highest robustness relative to climate change uncertainties when comparing to other flood mitigation measures such as resettlement, flood defences, erosion control, insurance, etc.

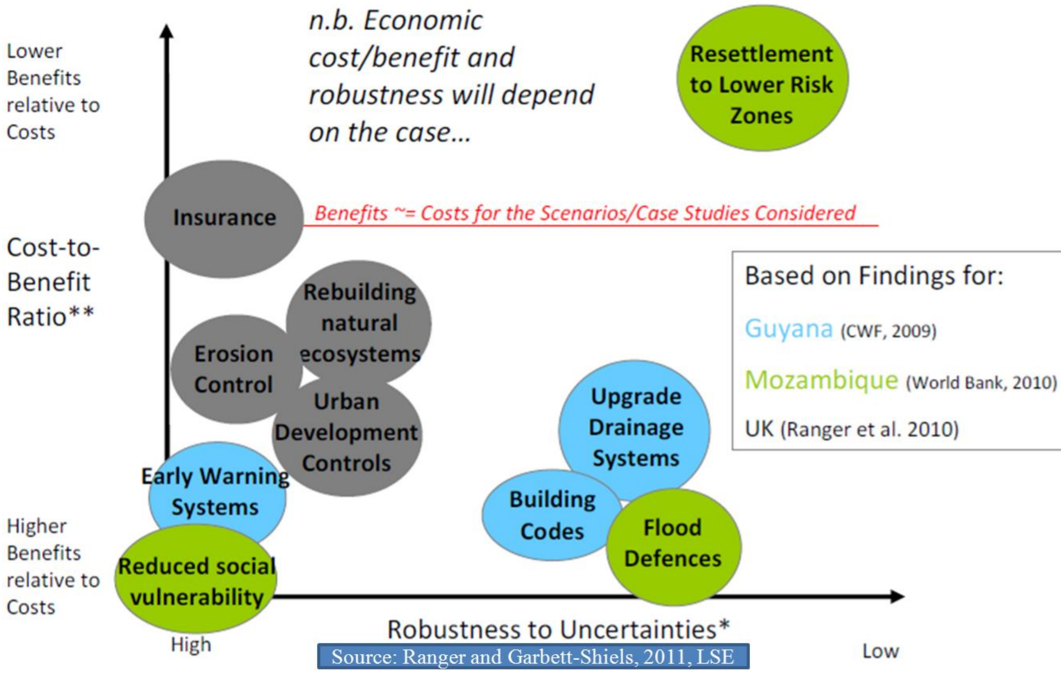
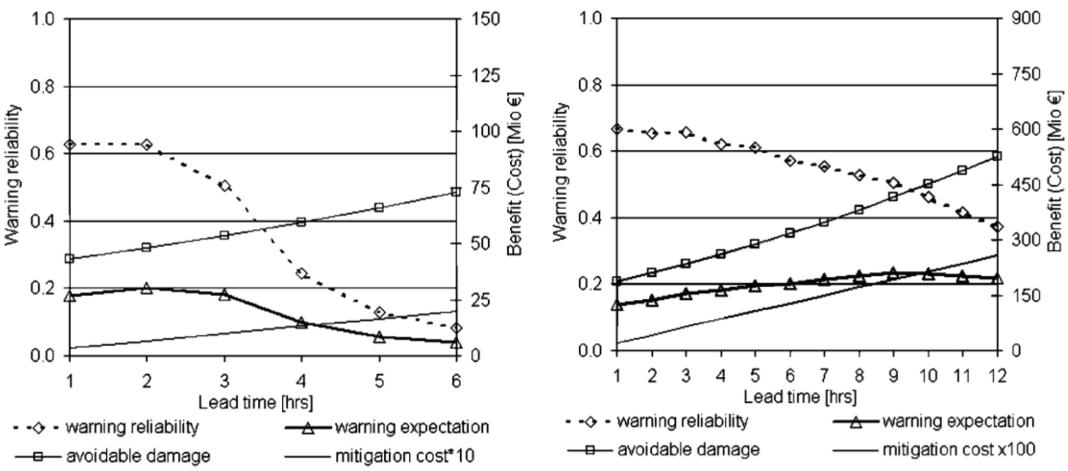


Figure 9.1. Benefits and uncertainties of flood management measures - FEWS included (Screenshot from the WBI e-institute Webinar, 3rd April 2012).

Warning systems are effective when they are accompanied by critical infrastructure – safe evacuation routes, shelters, etc. Studies have shown that damage reduction due to forecast improvements can range from a few percentage points to as much as 35% of average annual flood damages [205].

The efficiency of the FEWS is heavily based on the input data (forecast) and lead time of flood forecast. With increasing lead time, the FEWS models become less reliable but also, the avoidable damage increases. This is well described in reports [205-207]. Gocht et al.

[208] demonstrated how warning expectation is depended on the warning reliability, see Figure 9.2. The economic efficiency of disaster risk management is analysed by Mechler [209]. Nachtnebel conducted the assessment of the reliability and the efficiency of early flood warning systems [171].



a) Spain, Barcelona, Besos basin. b) Austria, Traisen basin.

Figure 9.2 Warning reliability and benefits over lag (lead time) [208].

Advantages as well as disadvantages of FEWS are discussed in SUFRI report [172].

Advantages of FEWS can be summarised as follows:

- the non-structural measure with best cost-benefit-ratio (Figure 9.1);
- provides information on floods before flood occurs with relatively accurate prediction up to 48 hours;
- ensures centralised data collection and supports the decision process.

Disadvantages of FEWS are:

- flood forecast cannot directly protect infrastructure, but can prevent indirect damage
- weather forecast and rainfall runoff models are very sensitive to inputs and depend on calibration (faults in the inputs or inadequate calibration can give false forecast);
- insufficient rainfall and river gauge monitoring network could result in poor forecast results;
- longer lead time increases uncertainty of the forecast;

9.4 Cost of Flood Forecast System - Prediction module

The total cost of prediction module is divided into (C.1) Cost required for modelling, (C.2) cost required for purchase, installation and maintenance of the equipment, (C.3) cost to automate data imports, running the models and model result exports for the flood forecasts, (C.4) costs of setting up the dissemination system flood warning and (C.5) running costs of FEWS including the annual maintenance of sensors, resources required for computations, data storage and maintenance of servers with required licences, cost of dissemination via SMS, email or twitter and minimum annual costs of resources (staff). The total cost of prediction module developed for Bandon Flood Forecasting system with a design duration of 2 years is estimated to be €67,049.83, as shown in Table 9.4. The cost breakdown is shown in Annex P.

Table 9.4 Cost of Bandon prediction module.

		Unit of Measure	Quantity	Unit price	Total
C	**Costs**				**€67,049.83**
C.1	Cost of modelling				**€10,500.00**
C.2	Installation of Monitoring equipment				**€28,274.91**
	No of Rain gauges	Quantity	3	€2,439.54	€7,318.61
	No of Water level Gauges	Quantity	4	€1,939.54	€7,758.15
	Monitoring at the bridge	Quantity	4	-	€13,198.15
C.3	Cost of Flood Forecast				**€10,500.00**
C.4	Cost of Flood Warning				**€6,750.00**
C.5	Running cost of FEWS	Years	2	€1,660.00	**€3,320.00**

** Recommended minimum requirements*

1 Rain gauge per	*200 km^2*
2 Water level gauges per junction	*2 per junction*
3 Water level gauges per km	*25 km*

Chapter 10
Summary and Conclusions

The literature review showed that bridge scour and hydraulic factors are the most common cause of bridge collapses worldwide [3, 5, 8-11]. Current bridge inspection methods are structurally oriented [1], and a lack of focus on the issue of scour was identified. Further, there is lack of standardisation in bridge inspection [1, 48, 49] meaning that any of existing methods are not easily transferable to other road or railway authorities. The need for adaptation of existing infrastructure to changing climate is already apparent, thus the importance of scour focused bridge inspections is even greater.

The results of this work successfully addressed the main theses that were set in section 1.3; these were:

Thesis 1:

"Introduction of a scour focused bridge inspection procedure would enhance existing structurally-oriented BMS and inspections".

Thesis 2:

"Introduction of Flood Forecasting System (FFS) will adapt BMS to climate change impacts."

The revision of existing bridge management systems (Chapter 3) and existing bridge scour inspection procedures (Chapter 4) yielded the conclusion that there is a need for standardisation and automation of scour inspection procedures. The upgrading of an existing scour inspection method, the Bekić-McKeogh method, referred to as B1 (see Annex I) was carried out in Chapter 5. The comparison between old (unstandardized) and newly developed scoring system for B1 showed low correlation ($R^2 = 0.38$). It was not possible to look for additional components in the available 100 reports which are based on method B1 to further improve B1 scoring system. As such, the new scoring system for Method B1 cannot be recommended for further use. Thus the development of a new method was recommended.

10.1 Solution for Thesis 1

The solution for the first thesis (*Thesis 1*) was achieved by developing, a new, standardised method for bridge scour inspections. This newly developed method can be introduced as an addition to any of the existing bridge structural inspections. It is standardised and transferable. The new bridge inspection method was split into two separate procedures – Level 1 and Level 2 inspection. The workflow and connection between the two procedures is described in detail and is explained in Figure 6.1 and Table 6.2. This separation reduces amount of time of the inspection for simpler structure(s) and reduces the overall cost of inspections in the bridge stock. The greatest cost reduction comes from lowering the cost of reporting (reporting time reduced between 96% - 98%) and the cost of River Bathymetry Survey (time for survey reduced by 50%). When compared to Method B1, the overall price per bridge inspection was reduced by 81%, from c. €760 to c. €145 for L1 bridge inspection. For Level 2, e.g. complex bridges, due to the need for a bathymetry survey, the reduction in cost was between 10-30%. The cost of Level 2 inspection was estimated to be in the range between c. €520 - €685. A detailed cost analysis was conducted in Chapter 9.

With more focussed procedure(s), the first method - Level 1 inspection is designed for simple bridges of single span. Method L1 requires a lower level training for the inspectors and has a more simple input data requirement. In this way it is possible to conduct bridge inspections in-house which can reduce costs even more. Method L1 is fully standardised, the decision is broken into components and the calculation, which is based on the combination of worst-component scenario and weighting factors is automated.

The second Level 2 inspection is designed for complex structures, e.g. Level 2 bridges of two or more spans. The inspection procedure requires collection of more detailed input data when compared to Level 1 inspection. The inspection usually comprises a site visit and sometimes office research. One of the most expensive parts of this type of inspection is a survey of the river bed and investigation of foundation depths. One of the advantages of this method is that it compares the scour depth with foundation depths, whereas for unknown foundations a conservative estimate of the depth of foundations is made. This makes this method superior to other methods available. In order to conduct L2 inspection, bridge inspectors need to undergo more detailed training (compared to L1 training) to gain experience in hydrology, hydraulics and bridge scour. Like method L1, method L2 is fully standardised and the inspector decision is broken into components. The mathematics for calculation of Scour Condition Rating (Sc.CR) is based on the combination worst-component scenario and pre-defined pair-wise lookup tables.

During the initial development of the mathematical language behind the methodologies, sensitivity analysis and testing (Annex D) of methods was conducted.

Furthermore, both of the inspection methods – Level 1 and Level 2 -- were applied on 44 and 100 bridges in Ireland respectively and verified against a more detailed and more time-consuming inspection methodology – Method B1 -- that was set as base method for comparison. The verification process was based on correlation analysis and comparison of the percentage of acceptable and unacceptable inspection results with results from method B1.

The analysis confirmed a strong correlation of $R^2 = 0.82$ between L1 and L2 inspection results compared to method B1 inspection results. Method L1 had zero unacceptable ratings when compared to method B1 on the sample of 44 bridges. Method L2 shows strong correlation with Method B1. There is a very low percentage of unexpected outcomes (4%) from the comparison of method L2 with Method B1. These unexpected outcomes were studied in more detail in case-by-case analyses and were proven to be more favourable for the method L2.

Methods L1 and L2 are recommended for an on-site application for the assistance of operational BMS. Weight factor(s) and lookup matrices are now verified. If a dataset with even larger sample of bridges becomes available in the future, further adjustment of the methods will be possible.

In case that during their application some undesirable calculation of Scour Condition Rating is noted, both methods can easily be further adjusted by changing the weighting factors in Method L1 or by adjusting the lookup matrices for Method L2.

10.2 Solution for Thesis 2

The solution for the second thesis (*Thesis 2*) was achieved by coupling a standard Flood Forecasting System (FFS) with bridge Scour Condition Rating. For the case study, an operational and fully automated FFS over Bandon river catchment (c. 600km^2) in Ireland was developed (see Chapter 8).

The system is capable of scheduling of bridge scour inspections up to 14 days in advance. The supervisor managing the bridge inspections does not need to rely on weather forecasts only as the system provides much more specific information such as water levels at the bridge location.

This thesis has demonstrated the Scour Depth Model (SDM), an unique model for any existing FFS as it is the only system known to the author that predicts exact scour depth at a bridge up to 14 days in advance and compares this information to the bridge foundations. The drawback of the SDM is that empirical formulas to predict scour depth very often overestimate theoretical scour when compared to actual scour [143-145].

Two practical examples (DSS1 and DSS2) for Decision Support Systems based on Flood Forecast were developed. DSS1 takes into account scour condition of the bridge (ScCR) and the flood level(s) at the bridge. DSS was applied on 101 railway bridges in Ireland. The system successfully demonstrates how the resources can be minimised during hydrological events of more frequent occurrence (≥50% AEP) and can be used in the most efficient way for flood events of lower probability of occurrence which are more dangerous to bridge safety from the bridge scour point of view. DSS1 also changes the approach for scheduling the time interval to the next bridge inspection. The interval between two bridge scour inspections in future could rely solely on the DSS1 model.

DSS2 is developed for bridge damage detection due to scour during extreme rainfall events. DSS2 is practical utilisation of Anderson et.al. [202] study that suggests the strong connection between the rainfall distribution and the extent of damage to bridges. The system is to be used for the crisis management for the most efficient allocation of inspection teams on site during the rainfall events that could threaten safety of the bridge.

Based on the ScCR, the DSS2 recommends action(s) as defined in Table 8.13. This system can be very valuable tool during storm events with significant amount of rain. DSS2 is cheaper than the DSS1 system, described in section 8.3.2, as it requires lower number of sensors (rain gauges or water level gauges) and relies on the existing rainfall forecasting products.

10.3 Recommendations and further work

Methods L1 and L2 are recommended for wide use on large amount of bridges. Standardisation of the Methods within national legislation should be explored. Research should focus on continuous updating and verification of weight factors (L1) and pair-wise lookup matrices (L2) based on additional datasets after methods are applied on even larger number of bridges.

Training is a very important part of successful implementation of the methods presented in this work. Integral part of training includes theoretical and practical part, e.g. office and field training. The training for Level 1 inspections is already in place and was delivered to the engineers in Cork Co. Co. in Ireland (Annex D). Training was additionally improved by introducing mobile application with integrated methods L1 and L2. Further training on the Level 2 guidelines needs to be set-up and presented to wider audience and bridge owners.

Development of BMS online platform and mobile Application for L1 and L2 is already underway. Further development and commercialisation of the platform and App that are based on work from this PhD is recommended.

Coupling of Flood Forecasting System (FFS) with bridge Scour Condition Rating (Sc.CR) was demonstrated and shown as a very useful and informative system for decision makers.

The FFS in BMS is one of the most apparent tools that enables adaptation of existing bridge stock for future more frequent and more extreme flood events. Two proposed models, DSS1 and DSS2, are fully transferable and recommended for application on any BMS with FFS in place. Further improvements and focus should be on elaborating and defining more clear actions (Table 8.12 and Table 8.13) that are recommended by

DSS1/DSS2. Research on this should be carried out by liaison between bridge managers, bridge inspectors, civil services and researchers. Training of staff and continuous simulations of events is recommended. Activity should also focus on development of training guidelines and handbooks with simulation scenarios.

The Scour Depth Module (SDM) should be equipped with scour monitoring sensors for verification. Real-time data of scour depth variations would be valuable information to assess and improve the existing scour depth empirical models. With further laboratory and on-site testing and on-site data gathering (water levels and scour depth monitoring) existing scour empirical equations and the proposed SDM reliability could be improved. These improvements of SDM could become an integral part of a Bridge Flood Forecasting System.

The benefits of FFS enable BMS adaptation to climate change impacts and weather extremes and provide added value to the new inspection methods presented and verified in this thesis.

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Chapter 11 Appendices

List of Annexes

Annex A Examples of bridge failures

As mentioned in sections 2.2.2 and 2.3.2, the cause of collapse of railway bridge Jakuševac in Croatia and the Malahide Viaduct in Ireland was due to least two scour types (one including a local scour). Below several examples of bridge collapses due to local scour are shown.

A.1 Cornwall bridge, Ontario, Canada, 1908

The Cornwall bridge is an example of bridge collapse caused by the heavy bank erosion of a lock (failed hydraulic structure upstream of the bridge) and a local erosion of the pier.

The bridge collapsed on two occasions, in 1898 and 1908. The first collapse occurred on 1st October 1898 during construction when a local scour of a pier occurred and two of three spans collapsed, causing 15 fatalities [210]. The second collapse occurred on 26th June 1908 due to a stone bank erosion at lock near the bridge [211]. The bank erosion progressed quickly and was around 60m wide several hours after the start of the leak. The huge amount of water caused scour and collapse of the pier.

Figure 11.1 Cornwall Canal after railway bridge collapse [211].

A.2 Old Bridge collapse, Bideford, Devon, England, 1968

The collapse happened after heavy rain and during a high tide during the night of 9th January 1968. According to article [212], the alarm was raised at Bideford Police Station by two women who had seen the arches fall. The extent of damage was not obvious as it was night-time. Catastrophe was prevented when a CID officer stopped a double-decker bus crossing the bridge. However, pedestrians were still allowed to walk over the bridge until the full extent of the damage became clear in daylight. The inspection showed collapse of arches and two piers due to river bed erosion (scour).

Figure 11.2 Partial collapse of Old Bridge in Devon, England, 1968 (Photo credit: Peter Christie)

A.3 Partial collapse of L7231 road bridge in Co.Cork, Ireland

In 2015, during a flash flood in Co. Cork, near Kinsale a partial collapse of a small stone arc bridge over a stream (tributary of Bandon River) occurred. The author of this thesis had the opportunity to witness the site and the extent of collapse. Both bridge abutments had experienced scour. The upstream arch barrel and retaining wall collapsed. Heavy undermining of the left bridge abutment was evident. The bridge was closed to local traffic and the repair of the bridge commenced the same year.

a) Upstream elevation of the bridge b) downstream elevation of the bridge

Figure 11.3 Partial collapse of small stone arch bridge in Co. Cork, Ireland.

A.4 Cumbria bridge collapses

Bridge collapses due to local scour from flash floods for two flood events from 2009 and 2015 will be listed.

A.4.1 Cumbria bridge collapses in 2009

In the flood event of 19th-20th November 2009 several bridge collapses occurred (Northside Bridge, Lorton Bridge, Calva Bridge, Navvies Footbridge, Camerton Footbridge, Memorial Gardens footbridge, and Little Braithwaite Bridge) [21, 213, 214]. Most of the collapsed bridges were masonry arch bridges. The causes of collapse of road bridges were reported to be sheer weight of the water and scour [214]. Debris in the flood waters helped cause the collapse of the smaller pedestrian bridges. More bridges were damaged during the flood. A rain gauge at Seathwaite Farm reported total rainfall of 316.4mm in 24 hours (the highest rainfall since beginning of records in Britain) [215]. Sibley [216] reports 403.4mm of rainfall in 38 hours.

A.4.1.1 Northside Bridge

Northside Road Bridge at Workington was the first to be swept away on 21st November 2009 resulting in the death of one person [214].

Figure 11.4 Northside bridge collapse (Photo credit: Simon Ledingham).

A.4.1.2 Camerton bridge

The Camerton bridge, over an old railway, located around 250m from the River Derwent collapsed due to flood waters in 2009. The flood extent was such that the river flooded its floodplain and created a river channel on the old railway line area, see grey area in Figure 11.5.

Figure 11.5 Camerton bridge collapse (Photo credit: Simon Ledingham).

A.4.2 Cumbria bridge collapses in 2015

The 2015 event gained a new record with a rainfall of 341.4 mm over a two-day period at Honister Pass rain gauge [215]. In Cumbria, over a dozen bridges and culverts collapsed with many bridges being temporarily closed pending inspection or safety-critical work for the repair of 205 bridges [217]. Several bridges: Cooley Bridge, Keswick Railway Path Bridge collapsed following heavy flooding and Camerton bridge collapsed six weeks after the flood.

A.4.2.1 Bell Bridge at Sebergham, River Caldew

As reported by Raymond [218], After the storm Desmond in December 2015, the 244-year old Bell Bridge was closed after post-flood cracks were found. The bridge collapsed six weeks later, on 27 January 2016 in a new storm named Jonas. The bridge had a significant historic value. The cost of design of a new bridge was £1.1m. The cause of collapse was flooding and abutment scour.

a) Prior the flood (photo credit: Sebergham Parish Council)

b) after the flood (photo credit: Kathryn Farrimond)

Figure 11.6 Bell bridge prior and after collapse (2016).

A.5 Bridge collapses in Co. Donegal (R. of Ireland) and Co. Derry, (Northern Ireland)

The flood event on 22/23 August 2020 occurred after a heavy rainfall of (33mm of rain at Malin Head) in just 2 hours, reported by Meteorologist Gerry Murphy to RTÉ (Ireland's National Television and Radio Broadcaster). As a consequence, substantial damage and collapse of several bridges occurred in Counties Donegal (Republic of Ireland) and Co. Londonderry (Derry) in Northern Ireland. On the 11th September 2017, RTÉ reported [219] that "Donegal County Council has estimated that the total repair cost for road infrastructure in Donegal following the recent floods is €15.3 million. Over 115 bridges were damaged in the course of the flooding, and the Council has said that up to a quarter of them may have to be replaced.". Climate Northern Ireland (NI) reported that more than 200 roads and 650 bridges in the region were damaged by the flooding. The NI Department for Infrastructure (DfI) estimated that the total repair bill would reach €11 million.

Collapsed bridge at Muff to Iskaheen road Photo: Niall Carson/PA Wire

Iniscarn Bridge, Co. Derry, N. Ireland (Photo: Margaret McLaughlin)

Collapsed bridge at Quigley's Point, Co. Donegal. Photo: Margaret McLaughlin

Bridge collapse near Claudy, N. Ireland Photo: Gary McCall

Figure 11.7 Photo documentation from Donegal/Derry August floods.

A.6 Hintze Ribeiro Bridge, Portugal, 2001

On the 4th of March 2001, after many days of intense rain and consequent increase of the river flows, scouring and collapse of one of the piers occurred. The collapse of the pier caused a partial collapse of the bridge deck which dragged down a bus and three cars, resulting in 59 fatalities. The report [98] states that "The collapse of the bridge, was later related to streambed scouring caused by illegal sand extraction", which suggests that the cause of the collapse might have been a combination of local and general scour. The tragedy showed the deficiencies of the bridge management carried out in Portugal and induced a re-design of existing bridge management, promoting campaigns for scour and underwater inspections in Portugal [98].

In an emergency response, from April to June of the same year a program for emergency inspections was launched. After 349 bridge inspections, three bridges were closed down and load/speed restrictions were enforced on 56 more.

Figure 11.8 Collapse of Hintze Ribeiro Bridge in Portugal [98].

Annex B Bridge Management Systems

Combining the research [1, 87, 88, 91, 92], total of 40 BMS in use are listed in table below.

Country	System Acronym and Authority
AUSTRALIA	MRWA (Main Roads Western Australia) NSW (Roads and Maritime Services (RMS), New South Wales)
AUSTRIA	BAUT (Brückendatenbank, BmfwA)
CANADA	OBMS (Ontario Ministry of Transportation and Stantec Consulting Ltd) QBMS (Quebec Ministry of Transportation) EBMS (Edmonton Ministry of Transportation) PEI BMS (Prince Edward Island Dept. of Transportation and Infrastructure) GNWT (Government of Northwest Territories, Department of Transportation)
DENMARK	DANBRO (Danish Bridges and Roads)
FINLAND	FinnRABMS (Finnish National Roads Administration Bridge Management System).
GERMANY	SIB-Bauwerke (German Highway Administration - BASt)
IRELAND	EIRSPAN (Transport Infrastructure Ireland)
ITALY	SAMOA (Surveillance, Auscultation and Maintenance of structures) APT-BMS (Autonomous Province of Trento)
JAPAN	MICHI (Ministry of Construction Highway Information Database) RPIBMS (Kajima Corporation and Regional Planning Institute of Osaka)
KOREA	KRMBS Korea Road Maintenance Business System (Korean Ministry of Land, Transport and Maritime Affairs)
LATVIA	Lat Brutus (Latvian State Road Administration)
NETHERLANDS	DISK (Dutch Ministry of Transport) COWI
NORWAY	BRUTUS (Norwegian Public Roads Administration)
POLAND	SMOK (Polish Railway Lines) SZOK (Local Polish Road Administrations)
PORTUGAL	SGOA - Sistema de Gestão de Conservação de Obras de Arte (SGOA)
SOUTH AFRICA	BMS.NRA (National Roads Authority) SIHA
SPAIN	SGP (Spanish Ministry of Public Works)
SWEDEN	BaTMan (Swedish Road Administrations, Swedish Railway Department)
SWITZERLAND	KUBA (Swiss Federal Roads Authority)
UK	STEG (Structures Register) HiSMIS (Highway Structures Management Information System) BRIDGEMAN (BRIDGE Management system) COSMOS (Computerized System for the Management of Structures)
USA	AASHTOWare (AASHTO) PONTIS (Preservation, Optimization and Network information System) BRIDGIT (Bridge Information Technology) PENBMS (Pennsylvania Bridge Management System) ABIMS (Alabama Department of Transportation) BridgeWatch®
VIETNAM	Bridgeman (Vietnam Ministry of Transportation)

Annex C Proposed Bridge Inventory

Proposed Bridge Inventory refers to a preferable amount of data needed to describe the bridge in order to place a bridge into a BMS.

1. GENERAL ADMINISTRATIVE INFO		DESCRIPTION	TYPE OF DATASET	UNITS
1	Geo-Region name	Geo-Region name	string	[1]
2	Country	Country name	string	[1]
3	Region	Region name	string	[1]
4	Sub-Region (State)	Sub-Region (State) name	string	[1]
5	County	County name	string	[1]
6	Time zone	Time zone of the region	string	[1]
DATA REQUIRED IN DATABASE:				
7	Country Boundaries	Country boundaries	.shp	[/]
8	Region Boundaries	Region boundaries	.shp	[/]
9	Sub-Region (State) Boundaries	Sub-Region (State) boundaries	.shp	[/]
10	County Boundaries	County boundaries	.shp	[/]

2. BRIDGE MANAGEMENT INFO		DESCRIPTION	TYPE OF DATASET	UNITS
11	Bridge Name	Name of the structure	String	[1]
12	Bridge ID	ID of the structure	String	[1]
13	Bridge Owner	Bridge Owner	Table	[1]
14	Maintaining Authority	Maintaining Authority	Table	[1]
15	Latitude	WGS84 Co-ordinates	numeric	degrees
16	Longitude	WGS84 Co-ordinates	numeric	degrees
17	Easting	Local coordinates	numeric	m
18	Northing	Local coordinates	numeric	m
19	Location map	Location map	.jpg, .png, etc.	NA
20	Bridge photo	Photo of bridge elevation	.jpg	[1]
OPTIONAL:				
21	Bridge drawing	Bridge drawing (if available)	.dwg, .pdf, .jpg	[1]
22	Year of construction	Year when the bridge was constructed (if available)	Integer	year
23	Year of reconstruction	Year when the bridge was reconstructed	Integer	year
24	Designer	Designer (if available)	String	[1]
25	Bridge Design Documentation	Bridge Designs (if available)	Link	[1]

3. BRIDGE GENERAL PROPERTIES		DESCRIPTION	TYPE OF DATASET	UNITS
26	Bridge Structure Classification	General classification of bridge structure	String	[1]
27	Number of spans	Total number of spans of the bridge	Integer	[1]
28	Width of Span 1 … Width of Span n	Width of spans	Numeric	[m]
29	Bridge founded on solid rock?	Bridge founded on solid rock (Yes or No)	(Y/N)	[1]
30	Bridge curved?	Bridge curved (Yes or No)	(Y/N)	[1]
31	Bridge Carriageway Skew	Skew angle of the carriageway to the bridge structure	Integer	degrees
32	Carriageway Approach Skew 1	Skew angle of Approach Carriageway 1	Integer	degrees
33	Width of Approach 1	Width of Approach Carriageway 1	Numeric	[m]
34	Carriageway Approach Skew 2	Skew angle of Approach Carriageway 2	Integer	degrees
35	Width of Approach 2	Width of Approach Road 2	Numeric	[m]
36	Vertical Clearance over carriageway	Vertical Clearance over carriageway	Numeric	[m]
37	Vertical Clearance under the bridge	Vertical Clearance under the bridge	Numeric	[m]
OPTIONAL:				
38	Load Capacity	Load Capacity	String	[1]
39	Design Load	Design Load	String	[1]
40	Load distribution class	Load distribution class	String	[1]
41	Assessment standards used	Assessment standards used	String	[1]

4. PRIMARY PASSAGE INFO		DESCRIPTION	TYPE OF DATASET	UNITS
42	Direction of Road	Direction of Primary Road	String	[1]
43	Roadside	Part of the road carried by bridge	Table	[1]
44	Over/Underbridge	Bridge is Overbridge or Underbridge	(Over/ Under)	[1]
45	Bridge Over Water	Bridge over water (Yes or No)	(Y/N)	[1]
46	River/stream name	Waterbody name (if applicable)	String	[1]
OPTIONAL:				
47	Annual Average Daily Traffic	Annual Average Daily Traffic	Integer	[1]
48	Percentage, light vehicles	Percentage, light vehicles	Integer	%
49	Percentage, heavy vehicles	Percentage, heavy vehicles	Integer	%
50	Direction of Road	Direction of Primary Road	String	[1]

Annex D Training for bridge inspections in Ireland

Title:	**Office and Field Training for Level 1 bridge scour inspection**	
Date:	Thursday 15th September 2016, 09:30 hrs	
Location:	CCC Road Design Office, Regional Training Centre, Innishmore, Ballincollig, Ireland	
Prepared by:	Igor Kerin, Kristina Potocki, Paul Cahill	
Attendance:	UCC	Eamon McKeogh, Igor Kerin, Paul Cahill
	UNIZAG	Kristina Potocki
	CCC	John Lapthorne, Linda Roberts, Daniel Ryan, Mark O'Sullivan, James Dwyer, Charlie McCarthy, Liam Dromey, Connor Larkin, Ken O'Riordan, Flor Rahilly, Eugene Finn, Brian Deasy

D.1 Background

Field Training was conducted, on 15th September 2016, by Igor Kerin and Eamon McKeogh for Cork County Council (CCC) engineers on two bridges:

a. Bridge A (Coolmucky brige)
b. Bridge B (Coolnagearagh Bridge)

At each location CCC engineers were divided in 3 groups (red, blue and green)+ group with trainee personnel (Igor Kerin, Paul Cahill and Kristina Potocki), and independently examined state of bridge elements following the spreadsheet form for inspection that is part of "Guidelines for Level 1 Bridge scour inspection". Each Group had a 3 members. The members of the groups were anonymous.

At the end of inspections scoring process and results are discussed. After all groups inspected the bridges, all participants were gathered and had general discussion of an inspection. Igor Kerin lead all participants through all components inspected and gave his recommendations and remarks.

CCC engineers gave their feedback on possible improvement of scoring process – regarding combining some elements in the same group.

D.2 Results

Results of states of the components of each groups were compared and discussed (see tables below). Results of all groups are showed that overall each group assigned the same Conditional Rating for the bridge A. Differences for some assigned states for the components for Bridge A were noted and discussed.

At the location of Bridge B (simple bridge) assigned states are almost identical for all groups. The blue Group assigned state C for Component L1.Sc.c06 which lead to the condition rating CR2. **It should be revised if the state C for the component L1.Sc.c06 should immediately lead to condition rating CR 2.** If state C of the component

L1.Sc.c06 leads to condition rating CR 2, then it should be assigned only in case if when it is evident that the bridge opening is seriously blocked by evident debris, all potential accumulation should have state B instead.

Trainees had generally had problems in identifying and recognising the component L1.Sc.c07 Embankment fill (e.g. slope, erosion etc) and it was difficult to distinguish difference between components L1.Sc.c08 and L1.Sc.c09 and between components L1.Sc.c10 and L1.Sc.c11.

During both site visits it is noted that components L1.Sc.c08 and L1.Sc.c09 can be merged into a single component to simplify the decisions during the inspections. Also, merging components L1.Sc.c10 and L1.Sc.c11 into a single component is recommended.

Bridge A (Coolmucky brige – CC-L2206-001.00) field visit inspection results

NO.	COMPONENT	GROUP / STATES ASSIGNED			
		Red	Green	Blue	Instructors
L1.SC.C01	Skew angle	C	C	C	B
L1.SC.C02	Location of bridge abutments	C	C	C	C
L1.SC.C03	Low deck / Possible pressure flow	C	C	B	C
L1.SC.C04	River bed slope in vicinity	C	C	B	C
L1.SC.C05	River bed material	C	C	C	C
L1.SC.C06	Debris accumulation potential	B	C	B	C
L1.SC.C07	Embankment fill (e.g. slope, erosion etc.)	C	D	C	A
L1.SC.C08	Scour state at the bridge (Section B)	D	C	D	-
L1.SC.C09	Protection state at the bridge (Section B)	-	D	-	D
L1.SC.C10	Scour state away from the bridge (Sections A,C)	D	D	D	-
L1.SC.C11	Protection state away from the bridge (Sections A,C)	-	D	-	D
	CONDITION RATING	Level 2	Level 2	Level 2	Level 2

Bridge B (Coolnagearagh Bridge) field visit inspection results

NO.	COMPONENT	GROUP / STATES ASSIGNED			
		Red	Green	Blue	Instructors
L1.SC.C01	Skew angle	C	C	C	C
L1.SC.C02	Location of bridge abutments	C	C	C	C
L1.SC.C03	Low deck / Possible pressure flow	B	B	B	B
L1.SC.C04	River bed slope in vicinity	B	C	C	B
L1.SC.C05	River bed material	A	A	C	A
L1.SC.C06	Debris accumulation potential	B	B	C	B
L1.SC.C07	Embankment fill (e.g. slope, erosion etc.)	A	A	A	A
L1.SC.C08	Scour state at the bridge (Section B)	-	A	-	-
L1.SC.C09	Protection state at the bridge (Section B)	A	A	A	A
L1.SC.C10	Scour state away from the bridge (Sections A,C)	A	A	A	A
L1.SC.C11	Protection state away from the bridge (Sections A,C)	A	A	-	-
	CONDITION RATING	CR 0	CR 0	CR 2	CR 0

Annex E List of maintenance and repair works

The following sections provides a breakdown of the works that can be requested for each bridge component. For each component of the bridge, a list of typical works is assigned during the bridge general or detailed inspection. This work can be carried out by the maintaining agent without the requirement of special inspections or design works.

E.1 St.c01 – Bridge Surface maintenance and repair works

Table 11.1 Maintenance module for St.c01 Bridge Surface

TYPE OF WORK	DESCRIPTION	UNIT
Replacement of asphalt pavement (dense type)	Resurfacing of footway or median with asphalt pavement (dense type) or similar approved material.	m²
Replacement of asphalt pavement (open type)	Resurfacing of footway or median with asphalt pavement (open type) or similar approved material.	m²
Replacement of surfacing course	Resurfacing of footway or median with surface course or similar approved material.	m²
Maintenance of concrete surface	Maintenance of concrete surfaces on the footway or median.	m²
Sealing of pavement cracks	Pavement cracks shall be sealed with a hot poured bitumen or similar approved product. The purpose of sealing these cracks is to prevent water ingress onto the deck of the structure.	m
Patching of potholes	Potholes present over or adjacent to structure shall be filled in with a macadam material or similar approved material.	m²
Road markings	Worn, faded or incomplete road marking on the wearing surface shall be repainted.	m
Maintenance of kerb stones	Disturbed, broken or misaligned kerbstones shall be repaired or relayed as appropriate.	m
Replacement of kerb stones	Disturbed, broken or misaligned kerbstones shall be replaced as appropriate.	m
Pavement remedial works	Pavement Remedial Works shall be carried out in accordance with appropriate national standards.	m²
Maintenance of paving	Relaying paving flags/ cobblestones on the footway or median.	m²
Sweeping and clearing	All debris, silt and vegetation shall be removed from the bridge surface, footway or median.	m²
Clearing of drain gullies	All drain gullies on or adjacent to structures shall be cleaned of silt, debris and vegetation.	no
Installation of drain gully	Installation of drainage gully on the bridge surface, with associated inlet and 10m tube.	No.
Rubbing strip	Installation of rubbing strip.	m²
Waterproofing	The replacement of waterproofing on the bridge deck.	m²
Miscellaneous works	-	item

E.2 St.c02 – Parapet maintenance and repair works

Table 11.2 Maintenance module for St.c02 Parapets

TYPE OF WORK	DESCRIPTION	UNIT
Removal of vegetation	All vegetation growth shall be removed from parapets.	m²
Repair of parapet	Any minor impact or other damages to parapets shall be made good.	m²
Cleaning & painting	Cleaning and painting of parapet/guardrail	m
Removal of graffiti	Graffiti shall be removed from all parapets.	m²
Masonry repointing	Masonry parapets with loose mortar shall be raked out, then all open joints shall be repointed with an appropriate masonry cement/sand mortar.	m²
Masonry repair	Damaged masonry parapets shall be repaired with similar stonework's and an appropriate masonry cement/sand mortar.	m³
Concrete repair	Repair of concrete sections of parapet.	m
Patch-painting of steel	Painted steel parapets showing evidence of minor corrosion shall be patch painted with an approved protective paint. Prior to painting, the surface shall be prepared as necessary.	m
Tightening of bolts	All loose bolt connections on steel and aluminium parapets shall be tightened.	
Maintenance of bedding mortar	Bedding mortar under metal parapet post baseplates shall be maintained to prevent standing water around the baseplate.	no.
Repair of parapet/guardrail	Damaged parapet posts and guardrails shall be replaced with a similar approved and compatible post or guardrail. All bolts shall be tightened to the required torque and anti-theft bolts fitted.	m
Replacement/ installation of guardrail	Damaged or missing guardrails shall be replaced with a similar approved and compatible guardrail.	m
Replacement of parapet	Damaged or missing parapets shall be replaced with a similar approved and compatible parapets, where no further repair works are required.	m
Replacement of parapet with repair of edge beam	Damaged or missing parapets shall be replaced with a similar approved and compatible parapets, and repair work performed on damaged edge beams of the bridge structure.	m
Miscellaneous works	-	item

E.3 St.c03 – Deck/ Slab/ Barrel maintenance and repair works

Table 11.3 Maintenance module for St.c03 Deck/ Slab/ Barrel

TYPE OF WORK	DESCRIPTION	UNIT
Cleaning of drip tubes	Drip tubes present on the soffit of decks shall be rodded clear.	no.
Concrete repairs	Repair of concrete sections of deck/ slab/ arch barrel.	m²
Hosing of surface	All growth (fungal, algal, etc.) on the deck/ slab/ arch barrel shall be removed by high pressure hosing.	m²
Removal of graffiti	Graffiti shall be removed from the deck/ slab/ arch barrel.	m²
Masonry repointing	Masonry arches with loose mortar shall be raked out, then all open joints shall be repointed with an appropriate masonry cement/ sand mortar.	m²
Masonry repair	Masonry arches with loss of section and mortar shall be repaired with similar stonework and an appropriate masonry cement/ sand mortar.	m³
Patch-painting of steel	Painted steel decks showing evidence of minor corrosion shall be patch painted with an approved protective paint. Prior to painting, the surface shall be prepared as necessary.	m²
Miscellaneous works	-	item

E.4 St.c04 – Beams and Girders maintenance and repair works

Table 11.4 Maintenance module for St.c04 Beams and Girders

TYPE OF WORK	DESCRIPTION	UNIT
Concrete repairs	Repair of concrete sections of beams or girders.	m²
Hosing of surface	All growth (fungal, algal, etc.) on the beams or girders shall be removed by high pressure hosing.	m²
Removal of graffiti	Graffiti shall be removed from the beams or girders.	m²
Patch-painting of steel	Painted steel beams or girders showing evidence of minor corrosion shall be patch painted with an approved protective paint. Prior to painting, the surface shall be prepared as necessary.	m²
Miscellaneous works	-	Item

E.5 St.c05 – Expansion Joints maintenance and repair works

Table 11.5 Maintenance module for St.c05 Expansion Joints

TYPE OF WORK	DESCRIPTION	UNIT
Cleaning of expansion joint	All dirt, debris and vegetation shall be removed from the expansion joints	m
Maintenance of expansion joint	Cracked, rutted, worn or delaminated asphaltic joints shall be repaired using appropriate materials and standards.	m
Miscellaneous works	-	item

E.6 St.c06 – Spandrel Walls/ Wing Walls/ Retaining Walls maintenance and repair works

Table 11.6 Maintenance module for St.c06 Spandrel Walls/ Wing Walls/ Retaining Walls

TYPE OF WORK	DESCRIPTION	UNIT
Removal of vegetation	All vegetation affecting the integrity of the Spandrel, wing or retaining walls shall be removed, including small trees growing from the wall, vegetation in motor joints and all vegetation within 1m of the wing/ retaining walls.	m^2
Concrete repair	Repair of concrete sections of spandrel, wing or retaining walls.	m^2
Hosing of surface	All growth (fungal, algal, etc.) on the spandrel, wing or retaining walls shall be removed by high pressure hosing.	m^2
Maintenance of soft joints	Any soft joints present on a wing wall shall be maintained. This may include replacing a polysulphide sealant with a similarly approved material.	m
Removal of graffiti	Graffiti shall be removed from all walls.	m^2
Maintenance of base protection	Base Protection at the base of the spandrel, wing or retaining walls shall be maintained to prevent water ponding at the base of the wall.	m
Masonry repair	Masonry spandrel, wing or retaining walls with loss of section and mortar shall be repaired with similar stonework and an appropriate masonry cement/ sand mortar.	m^3
Masonry repointing	Masonry spandrel, wing or retaining walls with loose mortar shall be raked out, then all open joints shall be repointed with an appropriate masonry cement/ sand mortar.	m^2
Miscellaneous works	-	item

E.7 St.c07 – Abutments maintenance and repair works

Table 11.7 Maintenance module for St.c07 Abutments

TYPE OF WORK	DESCRIPTION	UNIT
Removal Of Vegetation	All vegetation affecting the integrity of the abutments shall be removed, including small trees growing from the abutments, vegetation in joints and all vegetation within 1m of the abutments.	m^2
Maintenance Of Drainage Channel	The drainage channel on the bearing shelf and associated drainage outlets shall be cleaned and robbed to endure unimpeded flow of water from the bearing shelf.	m
Concrete Repairs	Concrete sections of abutments with minor damage as identified during inspections, such as minor spalling and cracking, shall be repaired.	m^2
Hosing Of Surface	All growth (fungal, algal, etc.) on the abutments shall be removed by high pressure hosing.	m^2
Maintenance Of Soft Joints	Any soft joints present on the abutment shall be maintained. This may include replacing a polysulphide sealant with a similarly approved material.	m^2
Removal Of Graffiti	Graffiti shall be removed from all abutments.	m^2
Maintenance Of Base Protection	Base Protection at the base of the abutment shall be maintained to prevent water ponding at the base of the abutments.	m^2
Masonry Repointing	Masonry abutments with loose mortar shall be raked out, then all open joints shall be repointed with an appropriate masonry cement/ sand mortar.	m^2
Masonry Repair	Masonry abutments with loss of section and mortar shall be repaired with similar stonework and an appropriate masonry cement/ sand mortar.	m^3
Establish Base Protection	Base protection shall be provided where there is evidence of water ponding around abutments, in the form of a sloped concrete apron.	item
Miscellaneous Works	-	item

E.8 St.c08 – Piers maintenance and repair works

Table 11.8 Maintenance module for St.c08 Piers

TYPE OF WORK	DESCRIPTION	UNIT
Removal of vegetation	All vegetation affecting the integrity of the piers shall be removed, including small trees growing from the piers, vegetation in joints and all vegetation within 1m radius from the piers.	m^2
Maintenance of drainage channel	The drainage channel on the bearing shelf and associated drainage outlets shall be cleaned and robbed to endure unimpeded flow of water from the bearing shelf.	item
Concrete repairs	Concrete sections of pier with minor damage as identified during inspections, such as minor spalling and cracking, shall be repaired.	m^2
Hosing of surface	All growth (fungal, algal, etc.) on the pier shall be removed by high pressure hosing.	m^2
Removal of graffiti	Graffiti shall be removed from all piers.	m^2
Maintenance of base protection	Base Protection at the base of the pier shall be maintained to prevent water ponding at the base of the piers.	m^2
Masonry repointing	Masonry piers with loose mortar shall be raked out, then all open joints shall be repointed with an appropriate masonry cement/ sand mortar.	m^2
Masonry repair	Masonry piers with loss of section and mortar shall be repaired with similar stonework and an appropriate masonry cement/ sand mortar.	m^2
Establish base protection	Base protection shall be provided where there is evidence of water ponding around the pier, in the form of a sloped concrete apron.	item
Miscellaneous works	-	item

E.9 St.c09 – Embankments maintenance and repair works

Table 11.9 Maintenance module for St.c09 Embankments

TYPE OF WORK	DESCRIPTION	UNIT
Removal of vegetation	All vegetation within a 1m radius of the structure should be removed and vegetation preventing access to components of the bridge, such as the substructure, should similarly be removed.	m^2
Maintenance of drainage channel	Existing drainage channels should be maintained and kept clear of any debris accumulation to ensure the swift removal of any surface water.	m
Maintenance of gabions	Any damaged gabions shall be repaired with similar wire to the original and missing stone infill replaced with a substitute similar to the original.	m^3
Maintenance of revetments	Any damaged or missing revetment protection, such as paving slabs, rock revetments, gabions, stone, in-situ concrete, etc., shall be repaired or replaced.	m^2
Reshaping and re-establishment of slope	Earth, or other imported material, embankments shall be repaired, re-shaped or re-profiled to its original slope profile, including repair of minor erosion.	m^3
Removal of graffiti	Graffiti shall be removed from all embankments	m^2
Miscellaneous works	-	item

E.10 St.c10 – Bearings maintenance and repair works

Table 11.10 Maintenance module for St.c10 Bearings

TYPE OF WORK	DESCRIPTION	UNIT
Cleaning of bearings	Bearings require cleaning of dirt and debris to prevent corrosion and deterioration.	no.
Maintenance of bedding mortar	Maintenance and repair of the bedding mortar of the bearings.	no.
Patch-painting of steel	Painted steel elements of the bearing showing evidence of minor corrosion shall be patch painted with an approved protective paint. Prior to painting, the surface shall be prepared as necessary.	m
Concrete repairs	Concrete repairs to the bearing and associated sections, such as the pedestal and shelf.	m^2
Miscellaneous works		item

E.11 Scour maintenance and repair works

Table 11.11 Scour maintenance works

TYPE OF WORK	DESCRIPTION	UNIT
Site clearance	General site clearance to gain access to the structure	m^2
Vegetation removal	Clearance of all vegetation including small trees on, attached to the bridge over the full surface area including killing of roots by injection with a suitable herbicide	m^2
Debris removal	Removal of all logs and branches other vegetation and other material that block the watercourse. All such material shall be removed such that none remains lodged against the substructure or embankments	m^3
Deposition of soil-fill	Soil deposition in eroded areas around bridge piers and abutments	m^3
Stone fill	Placement of missing rock and stone material in eroded areas and/or for filling soft spots on the riverbed and/or in revetment areas	m^3
Site clearance	General site clearance to gain access to the structure	m^2
Vegetation removal	Clearance of all vegetation including small trees on, attached to the bridge over the full surface area including killing of roots by injection with a suitable herbicide	m^2

Table 11.12 Scour repair works

TYPE OF WORK	DESCRIPTION	UNIT
Riprap	Riprap is placed in a river or coastal environment to prevent scour or erosion of the bed, banks, shoreline, or near structures such as bridge piers and abutments. It involves placement of rock and stone in layers on top of a bedding or filter layer composed of sand, gravel and/or geotechnical fabric.	m^3
Geotextile	Geotextile fabric used as a filter/separator beneath the riprap	m^2
Concrete grouting	Pressure grouting is used to fill voids below footings, where this is carried out underwater the grout can be injected using tremie pipes. Formwork is needed to contain the grout, which can take the form of concrete bags or sheet piles.	litre
Concrete skirts	Concrete walls precast or cast in place against the sides of footing extending below water level for protection and reinforcement of the foundation structure against erosion and hydraulic forces	m^3 (incl. reinforcement)
Bed lining	Reinforcement of the riverbed using different types of materials (e.g. puddle clay, plastic membranes, bentonite or bituminous geo-membranes, aprons, concrete	m^3 (incl. reinforcement)
Soil nailing	Reinforcing the soil with steel bars or other materials for stabilization of both natural slopes and vertical or inclined excavations that are in contact with the watercourse	nail

Annex F Selection of type of armouring

F.1 NCHRP/HEC-23 selection method

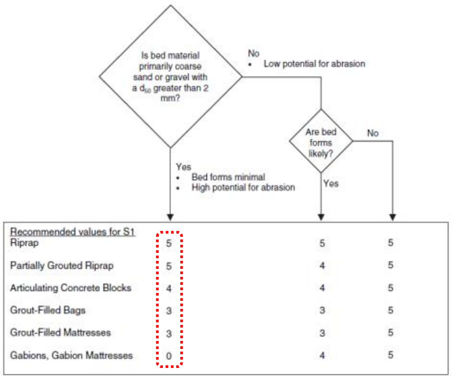
The selection method described in NCHRP [109] and HEC-23 [107, 108] report(s) presents the suitability of six armouring-type as Selection Index (SI) which is calculated from five influencing factors. The Selection Index (SI) is calculated from expression:

$$\mathbf{SI = (S1 \times S2 \times S3 \times S4)/LCC}$$

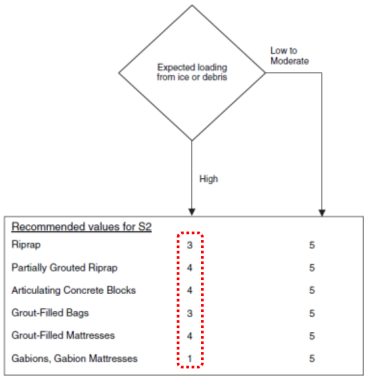
The following armouring-types are evaluated: standard (loose) rip-rap, partially grouted rip-rap, articulating concrete blocks, gabion mattresses, grout-filled mattresses, and grout-filled bags. The armouring-type that has the highest value of Selection Index (SI) is considered to be the most appropriate for a given site.

Five factors used to compute a Selection Index (SI) are: S1 - Bed material size and transport, S2 - Severity of debris or ice loading, S3 - Constructability constraints, S4 - Inspection and maintenance requirements, and LCC - Life-cycle costs. Influencing factors for this bridge are shown in Figure 11.9.

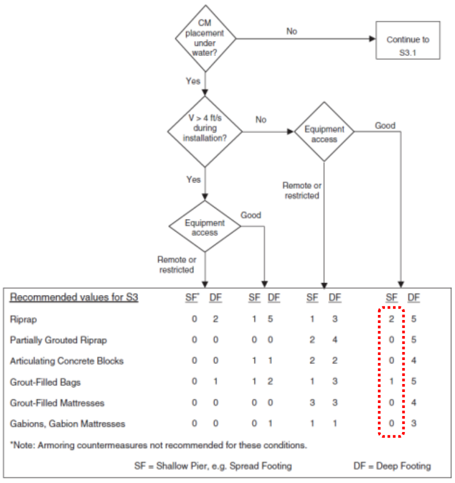
The Selection Index (SI) is sensitive to the life-cycle costs and assumptions regarding initial construction cost, remaining service life, assumed frequency of maintenance events, and extent of maintenance are required. In order to overcome possible misjudgement in life-cycle costs, they are assumed the same for all armouring-type, as suggested in the description of the method: *"It should be noted that the methodology can be used simply to rank the countermeasures in terms of suitability alone by assuming that the life-cycle costs are the same for all countermeasures."* (HEC-23 manual, page 3.5 [107])



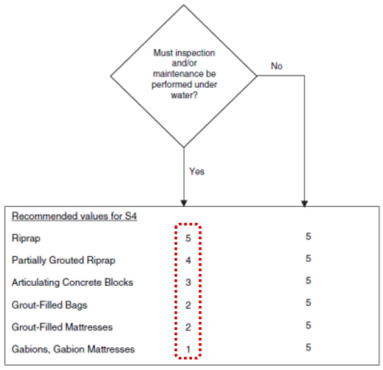
a) Factor S1: Bed material.



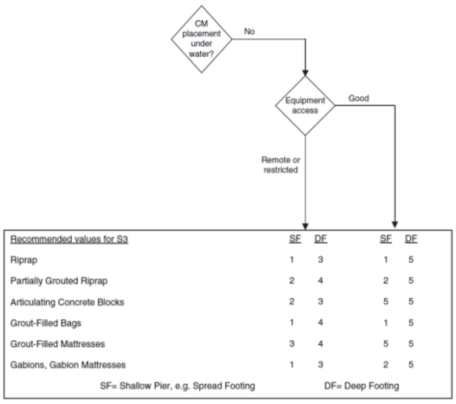
b) Factor S2: Ice/Debris load.



c1) Factor S3: Construction considerations.



d) Factor S4: Inspection and Maintenance.



c2) Factor S3.1: Construction considerations (No underwater placement).

Figure 11.9. Flowcharts of influencing factors [107, 108].

Two approaches in the life-cycle costs were used: Variant life-cycle costs and Constant life-cycle costs. *Variant life-cycle costs approach (a)* represent actual costs and uses different life-cycle costs (LCC) for each armouring type. The calculation of the LCC for each type of armouring was adopted from the TRB Excell spreadsheeds. For more accurate calculation of LCC, a detailed analysis is required. *Constant life-cycle costs approach (b)* ignores assumption on the LCC costs and uses the same LCC for each armouring type. Without consideration of life-cycle cost, the suitability of a countermeasure is dictated solely by the environment of the river and its interaction with the bridge structure, combined with the strengths and vulnerabilities of the countermeasure.

Table 11.13. Results of influencing factors.

Type of armouring	*S1 Bed Material*	*S2 Ice/ Debris Load*	*S3 Construction Considerations*	*S4 Inspection and Maintenance*	*(a) Variant LCC ($1000)*	*(b) Constant LCC ($1000)*
Rip-rap	5	3	2	5	$65.5	$50.0
Partially grouted rip-rap	5	4	0	4	$45.7	$50.0
Articulating concrete blocks	4	4	0	3	$63.9	$50.0
Grout-filled bags	3	3	1	2	$50.7	$50.0
Grout-filled mattresses	3	4	0	2	$41.3	$50.0
Gabions, Gabion mattresses	0	1	0	1	$61.2	$50.0

Table 11.14. Selection Index for variant and constant life-cycle costs.

LCC approach	*(a) Variant LCC*		*(b) Constant LCC*	
Type of armouring	*SI*	*Ranking*	*SI*	*Ranking*
Rip-rap	**2.3**	**1**	**3.0**	**1**
Partially grouted rip-rap	0.0	3	0.0	3
Articulating concrete blocks	0.0	3	0.0	3
Grout-filled bags	0.4	2	0.4	2
Grout-filled mattresses	0.0	3	0.0	3
Gabions, Gabion mattresses	0.0	3	0.0	3

F.2 CIRIA selection method

In the CIRIA method [4], the factors that influence the selection include:

- Underwater or dry construction
- Repairs
- Construction and maintenance costs
- Construction and maintenance constraints (low headroom, access)
- Environmental suitability
- Flow velocity
- Channel stability (laterally and vertically)

The general description of each influencing factor on the type of armouring is presented in a table matrix, as shown in Table 11.15.

As the channel of the Dodder River around the bridge is susceptible to vertical instability, and as the rigid systems cannot adjust to changes in the underlying surface, the rigid systems are excluded from further consideration for the UB63 bridge.

Table 11.15. Checklist for the influencing factors.

Legend	Factors								
● Good/appropriate ○ May be appropriate × Do not use/not applicable H High M Moderate L Low	Underwater construction	Repairs	Construction cost	Maintenance cost	Restricted access/headroom	Environmental suitability	High velocity flow	Vertical stream instability	Lateral stream instability
Flexible protection									
Rip-rap	●	●	L	M	○	●	●	●	○
Gabion mattresses and sacks	●/○	●	M	M	○	●	○	●	○
Gabion boxes	×	●	M	M	○	●	○	○	○
Articulated concrete blocks	○	●	H	M	○	○	○	●	○
Articulated grout-filled mattresses	●	●	M	L	●	○	○	●	○
Bituminous systems	×	●	L	M	○	○	○	●	○
Biotechnical solutions	○	●	M	M	○	●	×	●	○
Rigid protection									
Rigid grout-filled bags and mattresses	●	●	M	L	●	○	●	○	○
Concrete aprons	×	●	M	L	●	○	●	×	○
Stone pitching	○	○	M	M/H	○	○	●	○	○
Other									
Protective collars (piers only)	○	×/○	L	L	○	●	●	●	●
Pile caps/footings (piers only)	○	×/○	L	L	○	●	●	○	●
Sheet piling	●	●	M/H	L	○	●	●	●	●

Annex G "Raplab" tool for the design of rip-rap armouring

A number of different empirically derived formulae for rip-rap sizing have been developed over the years. Consequently, four equations (1) U.S. Army Corps of Engineers EM-1601, (2) Pilarczyk, (3) Escarameia and May, and (4) HEC-11 for sizing the rip-rap were used. As part of this thesis, a tool for design of the rip-rap is developed. The largest value of rip-rap size should be adopted as the design size, based on which design class should be assigned. Followed a guidelines from CIRIA [4] and HEC [51, 52, 107, 108] a tool called "Raplab" was developed as part of this thesis. The complete Matlab code is shown below with executable attached to a DVD copy of this thesis.

G.1 Input data

A tool requires following input data needed for the calculations of rip-rap mean diameter, as shown in Table 11.16. Note that the design flow velocity is obtained from the hydraulic assessmenr which usually requires development of a physical or hydraulic mathematical modells.

Table 11.16. Input data for sizing the rip-rap.

(y) - Local water depth:	[m]
(R) - Centerline radius of curvature of channel bend:	[m]
(W) - Width of water surface at upstream end of channel bend:	[m]
(SF) - Safety factor (must be >1):	[1]
(C_s) - Stability coefficient:	[1]
(C_T) - Blanket thickness coefficient given as a function of the uniformity ratio d85/d15:	[1]
(V_{mean}) - Channel cross-sectional average velocity:	[m/s]
(1V:mH) - Bank slope:	[1:m]
(Sg) - Relative buoyant density of the protection element:	[1]
(C_v) - Velocity distribution coefficient:	[1]

(ϕ_{SC}) - Stability correction factor:	[1]
(ψ_{CR}) - Mobility parameter:	[1]
(kt^2) - Turbulence factor:	[1]
(D_n) - Assumed diameter of rip-rap:	[m]
(x) - Roughness factor:	[1]
(β) - Longitudinal slope of the watercourse:	[°]
(ϕ) - Rip-rap angle of response:	[°]
(x_b) - Velocity factor:	[1]
(r) - Turbulence level:	[1]
(Sf_{HEC11}) - HEC-11 Stability factor level:	[1]

G.2 Rip-rap gradation

Once a design size is established, a standard size class can be selected from Table 11.17. These ten standard classes of rip-rap based on the median particle diameter d_{50} gradations were developed under NCHRP Project 24-23, "Riprap Design Criteria, Recommended Specifications, and Quality Control" (Lagasse et al. 2006) [220].

Table 11.17 shows gradations for ten standard classes of rip-rap which were developed under NCHRP Project 24-23, "Riprap Design Criteria, Recommended Specifications, and Quality Control". Table 11.18 shows particle weights in the rip-rap mixture. The proposed gradation criteria are based on a nominal or "target" d_{50} and a uniformity ratio d_{85}/d_{15} that results in rip-rap that is well graded. The target uniformity ratio d_{85}/d_{15} is 2.0 and the allowable range is from 1.5 to 2.5.

Table 11.17. Minimum and maximum allowable particle size.

		Diameter [mm]						
Nominal Rip-rap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		D_{100}
Class	Size [mm]	Min	Max	Min	Max	Min	Max	Max
I	150	90	130	140	180	200	230	300
II	230	140	200	220	270	290	360	460
III	300	190	270	290	360	390	470	610
IV	380	230	330	370	440	500	580	760
V	460	280	390	430	520	600	700	910
VI	530	330	470	510	610	700	830	1070
VII	610	370	530	580	700	790	940	1220
VIII	760	470	660	720	880	990	1170	1520
IX	910	560	800	860	1050	1190	1410	1830
X	1070	650	930	1020	1230	1380	1640	2130

Table 11.18. Minimum and maximum allowable particle weight.

		Weight [kg]						
Nominal Rip-rap Class by Median Particle Weight		W_{15}		W_{50}		W_{85}		W_{100}
Class	Weight [kg]	Min	Max	Min	Max	Min	Max	Max
I	9	2	5	7	12	18	29	64
II	27	6	18	23	41	59	100	214
III	68	15	42	55	95	141	232	500
IV	136	28	82	109	191	273	455	1000
V	250	50	141	186	327	477	795	1727
VI	375	77	227	295	523	750	1273	2727
VII	500	118	336	432	773	1136	1864	4091
VIII	1000	227	659	864	1500	2182	3636	8000
IX	2000	391	1136	1500	2636	3773	6318	13818
X	3000	614	1818	2364	4182	6000	10000	21909

In the section below, a Matlab code for the Raplab tool for the rip-rap design is shown. The tool is free to use and the author of the code is not responsible for any Third party misuse of the code or its results.

G.3 Matlab code for rip-rap design

```
%% Raplab is a tool for the design of rip-rap armouring
clear all
clc
%% Constants
g=9.81;
%% Log window for input parameters
prompt={'{\bf\fontsize{11}(y)    - Local water depth [m]:}',...
        '{\bf\fontsize{11}(R)    - Centerline radius of curvature of channel
bend [m]:}',...
        '{\bf\fontsize{11}(W)    - Width of water surface at upstream end of
channel bend [m]:}',...
        '{\bf\fontsize{11}(SF)   - Safety factor (must be >1) [1]:}',...
        '{\bf\fontsize{11}(Cs)   - Stability coefficient [1]} ...
(0.30 for angular rock 0.375 for rounded rock):'...
        '{\bf\fontsize{11}(C_T)  - Blanket thickness coefficient given as a
function of the uniformity ratio d85/d15 [1]}
(Ct=1.0 is recommended):'...
        '{\bf\fontsize{11}(V_[177]) - Channel cross-sectional average
velocity [m/s]:}'...
        '{\bf\fontsize{11}(m)    - Bank slope [1:m]:}'...
        '{\bf\fontsize{11}(Sg)   - Relative buoyant density of the
protection element [1]:}'...
        '{\bf\fontsize{11}(C_v)  - Velocity distribution coefficient [1]}
...                                    (1) - for straight channels or the
inside of bends (1.0); (2) - for the outside of bends (1.283 - 0.2log(Rc/W)
and 1.0 for Rc/W > 26); (3) - downstream from concrete channels (1.25); (4)
- at the end of dikes  (1.25 )'};
name='US-ARMY rip-rap design parameters';
numlines=1;
defaultanswer={'4','150','15','1.1','0.3','1',' ','1.5','2.65','2'};
options.Resize='on';
options.WindowStyle='normal';
options.Interpreter='tex';
input=inputdlg(prompt,name,numlines,defaultanswer,options);
clear prompt name numlines options defaultanswer
in=str2double(input);
%% Input water depth y
y=in(1,1);
if isnan(y)
    msgbox('Please enter Local water depth "y"', 'Error', 'error')
break
end
%% Input radius and width of the watercourse
rc=in(2,1);
if isnan(rc)
    msgbox('Please enter Centerline radius of curvature of channel bend
"rc"', 'Error', 'error')
break
end
W=in(3,1);
if isnan(W)
    msgbox('Please enter Width of water surface at upstream end of channel
bend "W"', 'Error', 'error')
break
end
%% Input safety factors, stability and thickness coeff.
Sf=in(4,1);
if isnan(Sf)
    Sf=1;
end
Cs=in(5,1);
if isnan(Cs)
```

```
    Cs=1;
end
Ct=in(6,1);
if isnan(Ct)
    Ct=1;
end
%% Input Average design velocity
Vsr=in(7,1);
if isnan(Vsr)
    msgbox('Please enter Channel cross-sectional average velocity "Vsr"',
'Error', 'error')
break
end
if rc/W<26
    Vproj=Vsr*(1.74-(0.52*log10(rc/W)));
elseif rc/W>26
    Vproj=Vsr;
elseif rc/W==26
    Vproj=Vsr;
end
%% Slope - 1:m shown in degrees
m=1/in(8,1);
if isnan(m)
    msgbox('Please enter Bank slope "m"', 'Error', 'error')
break
end
stupnjeva = atand(m) %tan-1(m)
%% Spec. weight of rip-rap
Sg=in(9,1);
if isnan(Sg)
    Sg=2.65
end
%% Velocity distribution coeficient
krb=in(10,1);
Cv2=1.283-(0.2*log10(rc/W)); %Koeficijent raspodjele brzine [1]
if krb==1
    Cv=1;
elseif krb==2
    Cv=Cv2;
elseif krb==3
    Cv=1.25;
elseif krb==4
    cv=1.25;
elseif isnan(krb)
    msgbox('Please enter Velocity distribution coefficient', 'Error',
'error')
break
end
%% Parameters for Pilarczyka
prompt2={'{\bf\fontsize{11}(\phi_{SC})  - Stability correction factor [1]:}
{\it(a) - exposed edges of gabions/stone matressess (1.0); (b) - exposed
edges of rip-rap and armourstone (1.5), (c) - continuous rock protection
(0.75), (d) - interlocked blocks and cabled blockmats (0.5)}',...
        '{\bf\fontsize{11}(\psi_C_R) - Mobility parameter [1]:}
{\it(a) - rip-rap and armourstone (0.035); (b) - box gabions and gabion
matresses (0.070); (c) - rock fill in gabions (<0.100)}',...
        '{\bf\fontsize{11}(k_t^2)   - Turbulence factor [1]:}
{\it(a) - normal turbulence level (kt^2 = 1.0); (b) - non-uniform flow,
increased turbulence in outer bends (kt^2 = 1.5); (c) - non-uniform flow,
sharp outer bends (kt^2 = 2); (d) - non-uniform flow, special cases (kt^2 >
2)}',...
        '{\bf\fontsize{11}(D_n) - Assumed diameter of rip-rap [m]}',...
        '{\bf\fontsize{11}(x) - Roughness factor}
{\it(k_s from 1 to 3)}',...
        '{\bf\fontsize{11}(\beta) - Longitudinal slope of the watercourse
[^o]:}',...
        '{\bf\fontsize{11}(\phi) - Rip-rap angle of response [^o]}'};
name='Pilarczyk rip-rap design parameters';
```

```
numlines=1;
defaultanswer={'0.75','0.035','1','0.38','1','2','40'};
options.Resize='on';
options.WindowStyle='normal';
options.Interpreter='tex';
input_Pilarczyk=inputdlg(prompt2,name,numlines,defaultanswer,options);
clear prompt name numlines options defaultanswer
in_Pilarczyk=str2double(input_Pilarczyk);
%% Pilarczyk parameter processing
phiSC=in_Pilarczyk(1,1);
if isnan(phiSC)
    msgbox('Please enter Stability correction factor (\phi_{SC})', 'Error',
'error')
break
end
psi=in_Pilarczyk(2,1);
if isnan(psi)
    msgbox('Please enter critical Mobility parameter of the protection
element (\psi)', 'Error', 'error')
break
end
kt2=in_Pilarczyk(3,1);
if isnan(kt2)
    msgbox('Please enter Turbulence factor (k_t^2)', 'Error', 'error')
break
end
Dn=in_Pilarczyk(4,1);
if isnan(Dn)
    msgbox('Please assume rip-rap diameter (D_n)', 'Error', 'error')
break
end
x=in_Pilarczyk(5,1);
if isnan(x)
    msgbox('Please enter roughness factor (x)', 'Error', 'error')
break
end
ks=x*Dn;
beta=in_Pilarczyk(6,1);
if isnan(beta)
    msgbox('Please enter Longitudinal slope of the watercourse (\beta)',
'Error', 'error')
break
end
phi=in_Pilarczyk(7,1);
if isnan(phi)
    msgbox('Please enter rip-rap angle of response ', 'Error', 'error')
break
end
%% Pilarczyk velocity profile
% Profil brzine Pitanje-odgovor
choice = questdlg('Define velocity profile factor', ...
    'Velocity Menu', ...
    'Fully developed logaritmic velocity profile','Not-fully developed
logaritmic velocity profile','Fully developed logaritmic velocity
profile');
% Handle response
switch choice
    case 'Fully developed logaritmic velocity profile'
        disp([choice ' selected.'])
        velprofile = 1;
    case 'Not-fully developed logaritmic velocity profile'
        disp([choice ' selected.'])
        velprofile = 2;
end
if velprofile==1
    kh=(2/(log10((1+(12*y)/ks))^2));
elseif velprofile==2
    kh=(((1+y)/Dn)^(-0.2));
```

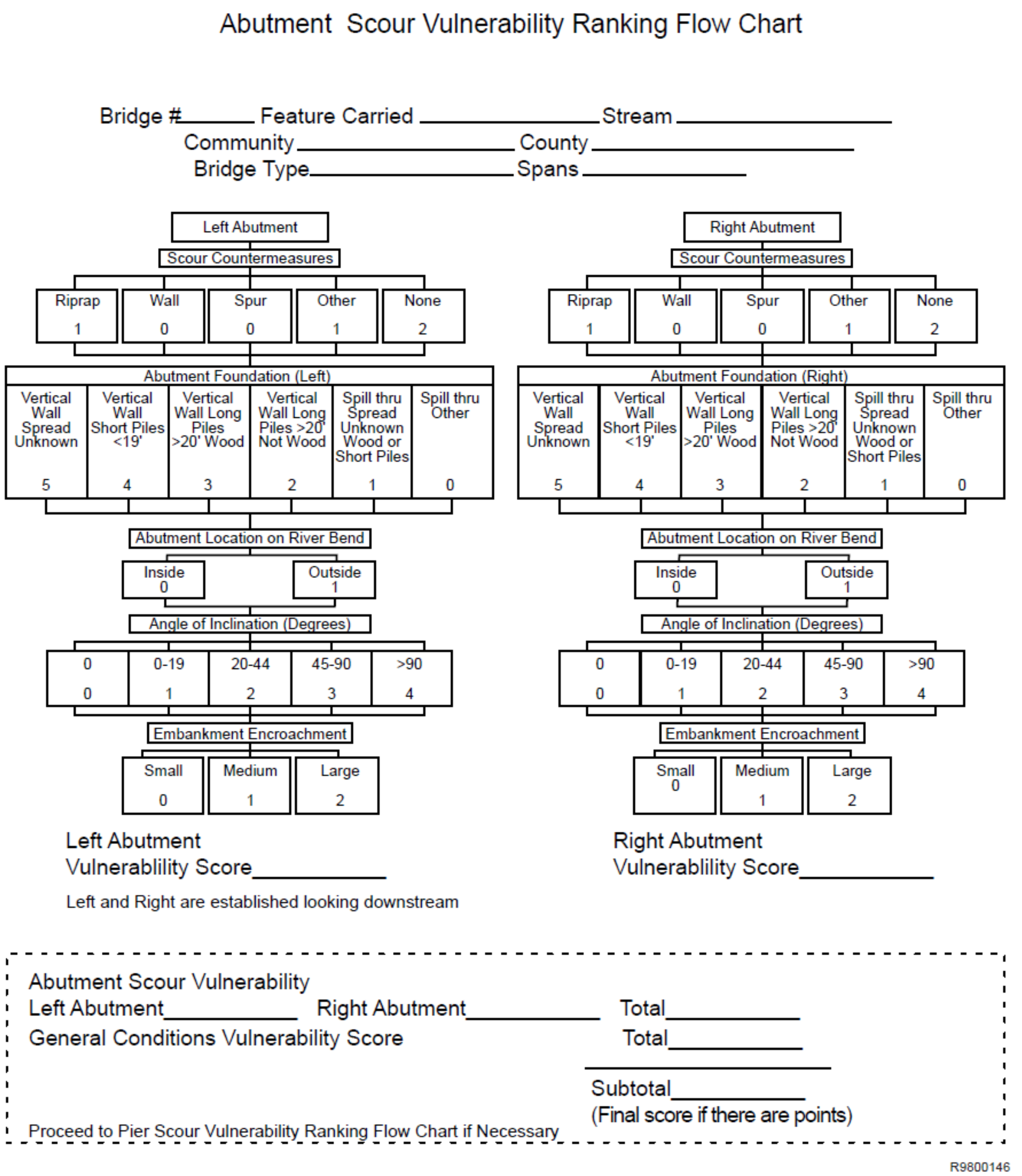
```
end
delta=Sg-1;
kd=sqrt(1-(sind(stupnjeva)^2/sind(phi)^2)); %side slope factor
kl=sind(phi-beta)/sind(phi);
ksl=kd*kl;

%% Escarameia & May
prompt3={'{\bf\fontsize{11}(x_b) - Velocity factor [1]:}
{\it recommended x_b = 0.74 to 0.90 U}'...
        '{\bf\fontsize{11}(r) - Turbulence level [1]:}
{\it(a) - straight river or channel reaches, normal (low) (r=0.12); (b) -
Edges of revetments in straight reaches, normal (high) (r=0.20); (c) -
Bridge piers, caissons and spur dikes and transitions, medium to high
(r=0.35 - 0.50); (d) - Downstream of hydraulic structures, very high
(r=0.60)}'...
        '{\bf\fontsize{11}(Sf_{HEC11}) - HEC-11 Stability factor level
[1]:}                                                {\it(a) - uniform
flow, Rc/W>30 (Sf_{HEC11} = 1.0 to 1.2; (b) - gradualy varying flow,
10<Rc/W<3 (Sf_{HEC11} = 1.3 to 1.6; (c) - rapidly varying flow, Rc/W<10
(Sf_{HEC11} = 1.6 to 2.0)}'};
name='HEC-11, Escarameia and May rip-rap design parameters';
numlines=1;
defaultanswer={'0.74','0.15','1','2'};
options.Resize='on';
options.WindowStyle='normal';
options.Interpreter='tex';
input_EM=inputdlg(prompt3,name,numlines,defaultanswer,options);
clear prompt name numlines options defaultanswer
in_EM=str2double(input_EM);

%% Escarameia & May parameters processing
xb=in_EM(1,1);
if isnan(xb)
    msgbox('Please enter Velocity factor (x_{b})', 'Error', 'error')
break
end
r=in_EM(2,1);
if isnan(r)
    msgbox('Please enter Turbulence level (r)', 'Error', 'error')
break
end
Sf_hec11=in_EM(3,1);
if isnan(Sf_hec11)
    msgbox('Please enter HEC-11 Stability factor level Sf_{HEC11})',
'Error', 'error')
break
end
ub=xb*Vsr;
ct_EM=12.3*r-0.2;

%% Calculation for US_ARMY_EM_1601
tic
K1=sqrt(1-((sind(stupnjeva-14)/sind(32))^1.6));
d30=y*Sf*Cs*Ct*Cv*((Vproj/sqrt(K1*(Sg-1)*g*y))^2.5);
US_ARMY_EM_1601=1.2*d30*1000;
%% Pilarczyk calculation
d_Pilarczyk=phiSC/delta*0.035/psi*kh/ksl*kt2*((Vsr)^2)/2/g;
d50_Pilarczyk=d_Pilarczyk/0.84*1000;
%% Calculation for EM
d50_EM=ct_EM*(ub^2)/2/g/delta*1000;
%% Calculation for HEC11
K1_hec11=kd;
Csg_hec11=2.12/(delta^1.5);
Csf_hec11=(Sf_hec11/1.2)^1.5;
d50_HEC11=0.00594*Csg_hec11*Csf_hec11*(Vsr^3)/sqrt(y)/(K1_hec11^1.5)*1000;
%% Record input data
myfolder = uigetdir;
f1 = fullfile(myfolder,'inputparameters.xls');
```

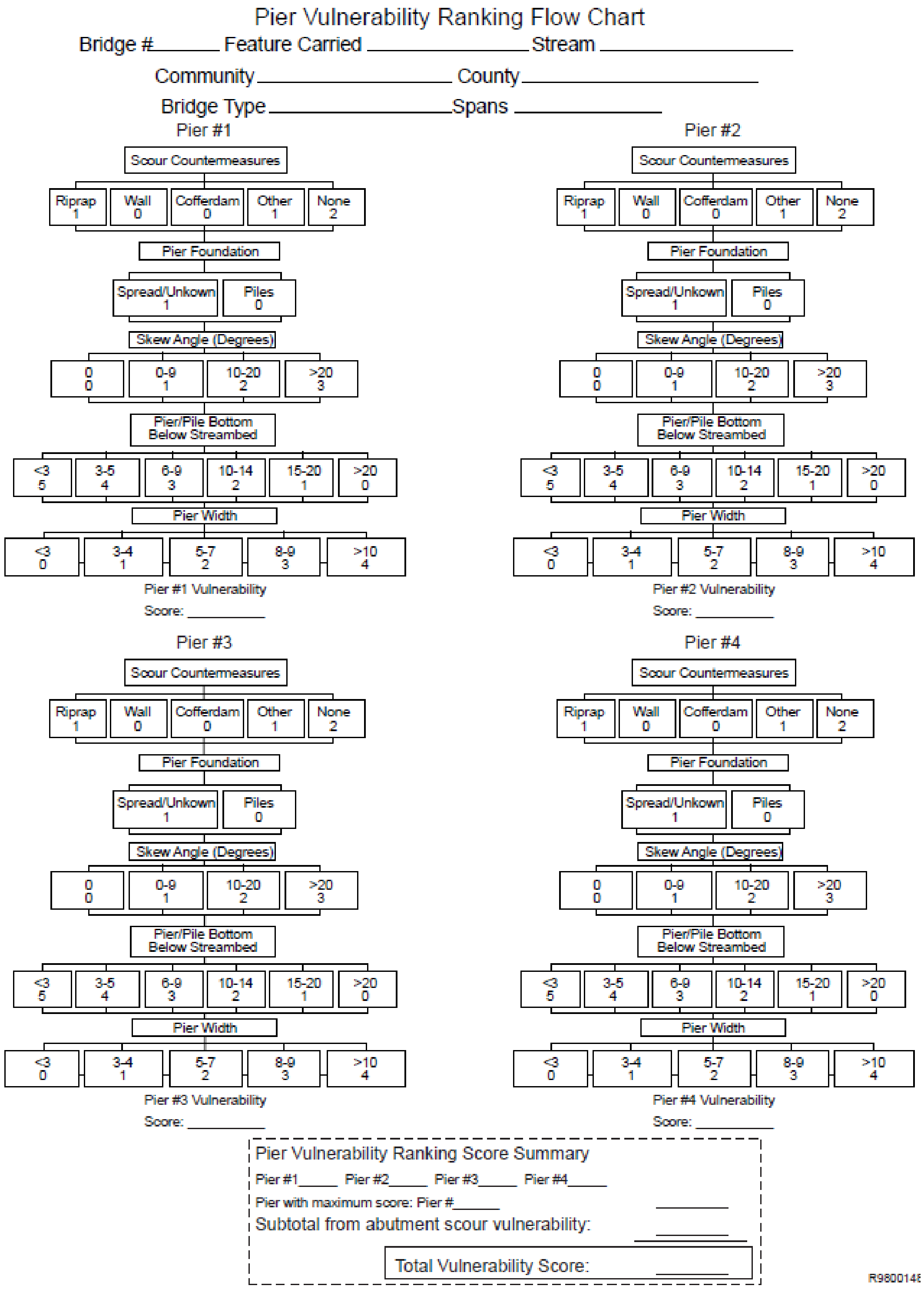
```
inpa={'(y)   - Local water depth [m]:';
        '(R)   - Centerline radius of curvature of channel bend [m]:';
        '(W)   - Width of water surface at upstream end of channel bend
[m]:';
        '(SF)  - Safety factor (must be >1) [1]:';
        '(Cs)  - Stability coefficient [1]:';
        '(C_T)  - Blanket thickness coefficient given as a function of the
uniformity ratio d85/d15 [1]:';
        '(Vsr) - Channel cross-sectional average velocity [m/s]:';
        '(m)   - Bank slope [1:m]:';
        '(Sg)  - Relative buoyant density of the protection element [1]:';
        '(C_v)  - Velocity distribution coefficient [1]:';
        '(phiSC)  - Stability correction factor [1]:';
        '(psiCR) - Mobility parameter [1]:';
        '(kt^2)   - Turbulence factor [1]:';
        '(Dn) - Assumed diameter of rip-rap [m]:';
        '(x) - Roughness factor [1]:';
        '(beta) - Longitudinal slope of the watercourse [o]:';
        '(phi) - Rip-rap angle of response [o]:';
        '(x_b) - Velocity factor [1]:';
        '(r) - Turbulence level [1]:'
        '(Sf_HEC11) - HEC-11 Stability factor level [1]:'};

inpall=[input(1,1);input(2,1);input(3,1);input(4,1);input(5,1);input(6,1);i
nput(7,1);input(8,1);input(9,1);input(10,1);

input_Pilarczyk(1,1);input_Pilarczyk(2,1);input_Pilarczyk(3,1);input_Pilarc
zyk(4,1);input_Pilarczyk(5,1);input_Pilarczyk(6,1);input_Pilarczyk(7,1);
        input_EM(1,1);input_EM(2,1);input_EM(3,1)];

    inpc=[inpa, inpall];

xlswrite(f1,inpc);

%% Graf
% Graphics plot Question-answer
choice = questdlg('Chose graph', ...
    'Graph Menu', ...
    'Normal scale on "x" and "y" axis','Logaritmic scale on "x" and "y"
axes','Logaritmic scale on "y" axis','Logaritmic scale on "x" axis');
% Handle response
switch choice
    case 'Normal scale on "x" and "y" axis'
        disp([choice ' coming right up.'])
        mjerilo = 1;
    case 'Logaritmic scale on "x" and "y" axes'
        disp([choice ' coming right up.'])
        mjerilo = 2;
    case 'Logaritmic scale on "y" axis'
        disp([choice ' coming right up.'])
        mjerilo = 3;
    case 'Logaritmic scale on "x" axis'
        disp([choice ' coming right up.'])
        mjerilo = 4;
end
xpl = 0.25:0.25:7; %korak
%US ARMY
if rc/W<26
    ypl_USARMY = y*Sf*Cs*Ct*Cv*(((xpl*(1.74-
(0.52*log10(rc/W))))/sqrt(K1*(Sg-1)*g*y)).^2.5)*1.2*1000;
elseif rc/W>26
    ypl_USARMY = y*Sf*Cs*Ct*Cv*((xpl/sqrt(K1*(Sg-1)*g*y)).^2.5)*1.2*1000;
elseif rc/W==26
    ypl_USARMY = y*Sf*Cs*Ct*Cv*((xpl/sqrt(K1*(Sg-1)*g*y)).^2.5)*1.2*1000;
end
%PILARCZYK
```

```
ypl_Pilarczyk =
(phiSC/delta*0.035/psi*kh/ksl*kt2*((xpl).^2)/2/g)/0.84*1000;
%EM
ypl_EM = ct_EM*((xb*xpl).^2)/2/g/delta*1000;
%HEC11
ypl_hec11=0.00594*Csg_hec11*Csf_hec11*(xpl.^3)/sqrt(y)/(K1_hec11^1.5)*1000;
%uvjeti max(ypl_USARMY,ypl_Pilarczyk,ypl_EM,ypl_hec11)
max1=max(ypl_USARMY);
max2=max(ypl_Pilarczyk);
max3=max(ypl_EM);
max4=max(ypl_hec11);

maxy=[max1 max2 max3 max4];

maxyy=max(maxy)+100;

if mjerilo==1
    plot(xpl,ypl_USARMY,':k^', xpl,ypl_Pilarczyk, '--ko',xpl,ypl_EM,'-
.kd',xpl,ypl_hec11,'-ks')
    set(gca,'XTick',0:0.5:7,'YTick',0:100:max(maxyy)) %x os od nula : s
korakom : do 7
elseif mjerilo==2
    loglog(xpl,ypl_USARMY,':k^', xpl,ypl_Pilarczyk, '--ko',xpl,ypl_EM,'-
.kd',xpl,ypl_hec11,'-ks')
elseif mjerilo==3
    semilogy(xpl,ypl_USARMY,':k^', xpl,ypl_Pilarczyk, '--ko',xpl,ypl_EM,'-
.kd',xpl,ypl_hec11,'-ks')
elseif mjerilo==4
    semilogx(xpl,ypl_USARMY ,':k^', xpl,ypl_Pilarczyk, '--ko',xpl,ypl_EM,'-
.kd',xpl,ypl_hec11,'-ks')
end
hlegend=legend('US-ARMY-EM-1601','Pilarczyk','Escarameia and May','HEC-
11');
title('Rip-rap median diameter relative to flow velocity');
xlabel('Flow velocity [m/s]');
ylabel('Median rip-rap diameter d_{50} [mm]');
set(hlegend,'FontAngle','italic','Location','Best')
%% Plot results
disp('rezultati:')
str1=['US_ARMY_EM_1601: ', 'd50= ', num2str(US_ARMY_EM_1601), '[mm]'];
disp(str1)
str2=['Pilarczyk: ', 'd50= ', num2str(d50_Pilarczyk), '[mm]'];
disp(str2)
str3=['Escarameia and May: ', 'd50= ', num2str(d50_EM), '[mm]'];
disp(str3)
str4=['HEC-11: ', 'd50= ', num2str(d50_HEC11), '[mm]'];
disp(str4)
msgbox({num2str(str1);num2str(str2);num2str(str3);num2str(str4)},
'Rezultati:')

%% Record output results
f2 = fullfile(myfolder,'DesignDiameters.xls');
inpa2={'US ARMY - EM 1601 d50 =';
        'Pilarczyk d50 =';
        'Escarameia and May d50 =';
        'HEC-11 d50 ='};
inpall2={US_ARMY_EM_1601; d50_Pilarczyk; d50_EM; d50_HEC11};
unit = {'mm';'mm';'mm';'mm'};
inpc2=[inpa2, inpall2, unit];
xlswrite(f2,inpc2)
%% End
toc
disp('***Normal run complete***')
clock
```

Annex H Colorado Scour Vulnerability Ranking Flow Charts

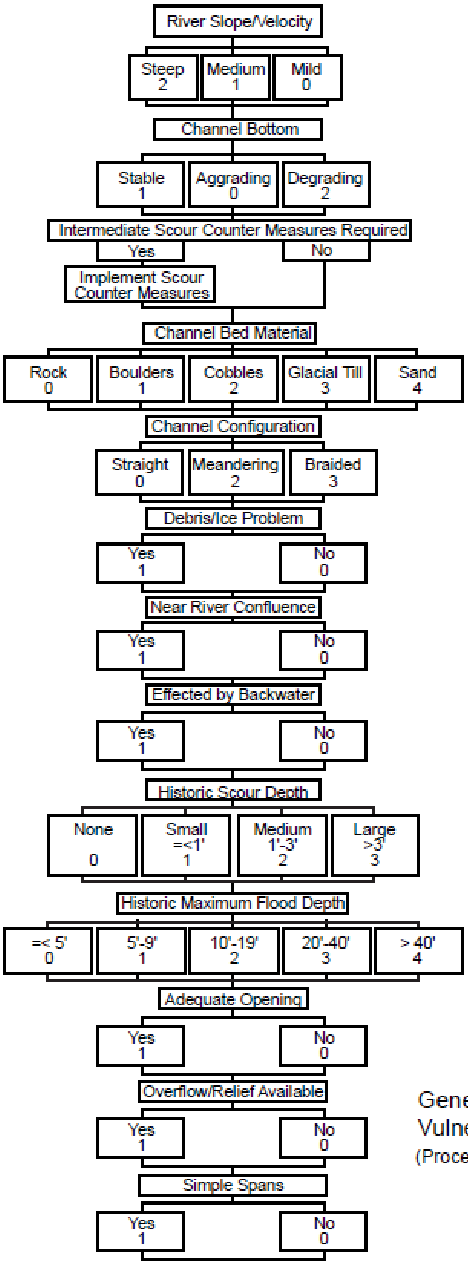
Colorado Bridge Safety Assurance Procedure for Colorado Highway Department Scour Vulnerability Ranking Flow Charts




Pier Vulnerability Ranking Flow Chart
Bridge # ______ Feature Carried ______ Stream ______
Community ______ County ______
Bridge Type ______ Spans ______
Pier #1
Scour Countermeasures
Riprap 1
Wall 0
Cofferdam 0
Other 1
None 2
Pier Foundation
Spread/Unkown 1
Piles 0
Skew Angle (Degrees)
0 0
0-9 1
10-20 2
>20 3
Pier/Pile Bottom Below Streambed
<3 5
3-5 4
6-9 3
10-14 2
15-20 1
>20 0
Pier Width
<3 0
3-4 1
5-7 2
8-9 3
>10 4
Pier #1 Vulnerability Score: ______
Pier #2
Scour Countermeasures
Riprap 1
Wall 0
Cofferdam 0
Other 1
None 2
Pier Foundation
Spread/Unkown 1
Piles 0
Skew Angle (Degrees)
0 0
0-9 1
10-20 2
>20 3
Pier/Pile Bottom Below Streambed
<3 5
3-5 4
6-9 3
10-14 2
15-20 1
>20 0
Pier Width
<3 0
3-4 1
5-7 2
8-9 3
>10 4
Pier #2 Vulnerability Score: ______
Pier #3
Scour Countermeasures
Riprap 1
Wall 0
Cofferdam 0
Other 1
None 2
Pier Foundation
Spread/Unkown 1
Piles 0
Skew Angle (Degrees)
0 0
0-9 1
10-20 2
>20 3
Pier/Pile Bottom Below Streambed
<3 5
3-5 4
6-9 3
10-14 2
15-20 1
>20 0
Pier Width
<3 0
3-4 1
5-7 2
8-9 3
>10 4
Pier #3 Vulnerability Score: ______
Pier #4
Scour Countermeasures
Riprap 1
Wall 0
Cofferdam 0
Other 1
None 2
Pier Foundation
Spread/Unkown 1
Piles 0
Skew Angle (Degrees)
0 0
0-9 1
10-20 2
>20 3
Pier/Pile Bottom Below Streambed
<3 5
3-5 4
6-9 3
10-14 2
15-20 1
>20 0
Pier Width
<3 0
3-4 1
5-7 2
8-9 3
>10 4
Pier #4 Vulnerability Score: ______
Pier Vulnerability Ranking Score Summary
Pier #1_____ Pier #2_____ Pier #3_____ Pier #4_____
Pier with maximum score: Pier #______ ______
Subtotal from abutment scour vulnerability: ______
Total Vulnerability Score: ______
R980014

General Conditions
Scour Vulnerability Ranking Flow Chart

Bridge #_____ Feature Carried __________ Stream __________
Community __________ County __________
Bridge Type __________ Spans ________



General Condition
Vulnerablility Score __________
(Proceed to Abutment Scour Vulnerability Ranking Chart)

R9800145

Annex I Bekić-McKeogh Method B1

The scope of Stage 1 – Qualitative Assessment is to identify important hydrological and hydraulic characteristics of combined interaction between the bridge design and the watercourse. The main aim of this level of bridge inspection is to identify those bridges where the risks are significant and remedial action needs to be taken. The assessment will rely primarily on the judgment of the Inspector carrying out the evaluation. It should be stressed that Stage 2 Analysis should proceed unless it is clear that a bridge can be considered safe from scour.

Hydrological/hydraulic factors that are considered are:

- characteristics of catchment (fluvial or estuarine),
- upstream flow conditions,
- bridge geometry, and
- downstream flow conditions.

The main deliverables of the Bridge Hydraulic Inspections are:

- Priority Rating for the bridge scour potential
- Years to next inspection
- Recommendations

I.1 Priority Rating

The Priority Rating is an indication of the relative potential for scour damage and need for further consideration and possible action. The method does not provide a quantitative assessment of the risk of failure, and no implications should be drawn from the Priority Rating regarding absolute values of risk. A quantitative evaluation of the scour potential is a part of the next stage of Bridge Scour Programme.

Table 11.19. Modified BA74/06 priority ratings.

Priority rating (PR)	Rating
Insignificant risk	1
Low risk (maintenance, minor	2
Move to Stage 2 - Analysis	3
Immediate action required (PoA)	4

Priority rank 1 - "Insignificant risk" implies that scour risk is minimal and the next bridge inspection is recommended after 6 years. Priority rank 2 - "Low risk" is assigned to bridges for which there might be some potential of developing one or a combination of three types of scour (general, local or constriction) at the bridge or around the bridge, but at the time of the inspection scour risk is acceptable. When assigning PR 2 recommendations with following actions at the bridge and/or around the bridge are listed. One of possible recommendations is bridge monitoring depending on parameter that needs to be monitored and which could affect bridge safety in foreseeable future. Priority rank 2 follows a special recommendation for the next bridge inspection which is within range from 1 to 5 years. Recommendation for next bridge inspection (ranging from 1 to 5 years) allows additional flexibility in ranking the bridges. In some cases it is difficult for inspector to decide if the bridge is at "low risk" or a next step "*Move to Stage 2 – Analysis*" is more appropriate. Although bridge safety is always a first criteria, in certain cases it is justified (from economical point of view) to assign shorter interval to next bridge inspection (1-2 years).

I.2 Years to next inspection

Recommendation for next bridge inspection (ranging from 1 to 5 years) allows additional flexibility in ranking the bridges. In some cases it is difficult for inspector to decide if the bridge is at "low risk" or a next step "*Move to Stage 2 – Analysis*" is more appropriate. Although bridge safety is always a first criteria, in certain cases it is justified (from economical point of view) to assign shorter interval to next bridge inspection (1-2 years).

I.3 Current bridge status

In this step data, known problems, conclusions and recommendations from previous reports and documents are highlighted.

I.4 Bridge Layout Inspection and Flows

In two steps (2.1 and 2.2) evaluation of global scour potential and catchment / river hydrology are accessed. The step 2.3 evaluates if flow conditions around the bridge differ from global flow conditions, hence if bridge construction obstructs the flow during the low and the flood flows.

I.5 Channel stability

This evaluation can be obtained even without bridge site visit because it evaluates global (not local) characteristics of the stream/river. The main objective of this step is to assess the channel stability and general scour potential (lateral and/or vertical) of a stream near a bridge from observable characteristics of the site. The conclusions can be made based on the following information.

I.5.1 Information from maps and satellite images

In order to collect information one should use satellites images and historic maps from "www.osi.ie" or "www.gsi.ie", or GoogleEarth, or any other useful source.

I.5.1.1 Find orthophoto of the bridge layout.

i. Describe if a bridge is located on the straight or curved section of the river. Define the relative bridge location referring to the longitudinal division (see Figure 2.4), i.e. bridge is located on upper, middle or lower (estuary/delta) part of the river:
ii. Note three river sections relative to the bridge location. The first section is upstream of the bridge. The second section is around the bridge (even up to a few tens of kilometres). The third section is downstream of the bridge. Describe characteristics of each of the three sections, as follows:

- general stream characteristic
  - evaluate if the section is a natural watercourse or it is altered by man
  - describe condition of banks and type of embankments, or just determine if there is an embankment
- floodplain characteristic (land use, evidence of floodplain flow)
- river channel slope
  - is it steep ($S>0.0015$ m/m) or medium ($0.0004<S<0.0015$ m/m) or mild ($S<0.0004$ m/m), which could be assessed from looking at terrain altitude and section length
- Is the stream around the bridge a navigational channel
- look up for constructions in the river channel (weirs, locks for ships, etc.)
- look up if there are nay tributaries around the bridge, also note if the river/stream which bridge is crossing is a tributary of another stream in the vicinity of the bridge
- lateral and vertical stability
  - look for evidence of morphological activity (point bars, braided river, sedimentation, etc.)
  - Lookup and compare existing and historic maps of the river channel. Comparison of the river channel conditions between existing and

earlier pre-existing condition could be a proof of lateral and/or vertical instability of the river channel. It should be noted if these changes are natural or due to human interventions (straightening of the channel). All data which is considered to be important should be identified. Any significant change must be noted and commented.
  - o vertical stability is almost impossible to determine from orthophoto and will be estimated if there is historic or field data)

I.5.2 Other information (books, reports, texts, drawings, internet)

Try to investigate any historical or field morphological data:

I.5.2.1 Geometry: Cross-sections or plans with river channel depths

General scour at a specific period in time can be measured by determining the difference in bed elevation between pre-flood and flood measurements of uncontracted cross sections; however, measurements of uncontracted cross sections during floods are rarely available. Comparing older and newer river bed elevations we can examine if there are some significant changes (degradation or even aggradation) of the river bed. Contracted sections should not be used because the scour measurements based on these sections will include contraction scour, in addition to the short- and long-term scour components.

I.5.2.2 Geomorphology (age and changes) and soil of the riverbed

Lookup for recorded changes and evident historic erosion / scour related problem for the stream and sediment transport in the river.
The water, as it flows over the channel bed, is able to mobilize sediment and transport it downstream, either as bed load, suspended load or dissolved load. The rate of sediment transport depends on the availability of sediment itself and on the river's discharge.
Rivers are also capable of eroding into rock and creating new sediment, both from their own beds and also by coupling to the surrounding hillslopes. Data that are of interested include:

- Stratification of the soil
- Characteristic of the layers (soil erodability)
- Grain-size curve of the riverbed material
- Median grain size D_{50}

Soil profiles around the bridge may not be representative samples for determining of the general vertical stability of the stream as the soil could be changed during the bridge construction. Soil samples around the bridge would be used in constriction and local scour depth.

Table 11.20. Typical Bed Material Characteristics [127],

Terrain	Channel slope	Typical bed material	Typical median grain size [mm]
Mountainous	Steep	Boulders, cobbles, gravels, sands	10
Upland	Moderately steep	Cobbles, gravels, sands	5
Hilly	Moderate	Gravels, sands	2
Lowland	Flat	Sands, silts and clays	0.5
Estuary	Varies with tide	Sands and silts	0.1

I.5.2.3 Geology

Lookup for geology maps around the bridge location which could indicate bedrock at bridge site.

I.5.2.4 Other bridges

Lookup for evidence of historic bridge collapses or scour related problems of other bridges in the vicinity / at the same catchment/stream/river of as bridge which is under inspection.

I.5.2.5 Any other recorded characteristic of the stream

Look for any information about river channel instability.

I.6 Extreme flows

This step comprises of stream and basin hydrology (Rainfall Runoff) data collection. The purpose of this investigation is to determine if there are occurrences of flooding around the bridge and if any at which extent. It is important to determine if the floodplains would be activated during the flood events as this information will be used for evaluation of constriction scour potential (chapter I.7). In this step following questions need to be answered:

1. Determine type of the river/stream around the bridge
    - Is the river/stream fluvial?
    - Is the river/stream tidal?

2. Is there a history of flooding in the vicinity of the bridge and is there evidence of the bridge collapses in the vicinity of the bridge
    - Flood maps (http://floodmaps.ie), reports, photographs, newspapers, residence experience, etc.

3. Floodplain landuse:
    - Land use (agriculture, urban, forest, etc.)
    - Are floodplains prone to flooding
    - If flooded are floodplains active (higher flow velocities)

4. Gather data on water levels and flow rates[14] from gauging stations in the vicinity of the bridge. Hydrograph characteristics are to be analysed (with sudden/steep changes, with mild changes, intermittent hydrograph, peak flows and water levels, etc.).

I.7 Constriction to the flow

Based on gathered data from previous steps it is estimated to what extend the bridge design constricts the flow causing flow acceleration (increased flow velocities) at the bridge. There are four main questions that need to be answered in this step:

1. Does the bridge construction constrict the flow during low and medium flows?

Procedure for answering above question is described in flow chart showed in Figure 11.10. This step is valid while flow is within the main channel and it needs to be checked before checking constriction during the flood flows as constriction scour might occur even during low flows.

2. Does the bridge construction constrict the flow during flood flows?

After answering 1st question check of the constriction flow for flood flows follows. This procedure is independent on evaluation low and medium flow constriction scour potential. Procedure for answering above question is described in flow chart showed in Figure 11.11.

[14] Although Stage 1 is a qualitative procedure/evaluation, based on flow rates and bridge opening (flow area) it is possible to roughly evaluate flood flow velocities around the bridge which could be indicative in determing potential for constriction scour.

3. **Is there a risk of accumulation of floating debris at the bridge soffit or around the bridge piers which would reduce flow area and constrict the flow?**

Debris accumulation around bridge abutment and piers or even at bridge soffit (during extreme flood events) could cause flow acceleration around the bridge. During the bridge inspection it is possible to determine if there is potential for floating debris accumulation at the bridge construction (in some cases debris accumulation is apparent during the bridge inspection). Indications of floating debris accumulation could be:

- Debris accumulation during the bridge inspection,
- Instability of upstream river banks which are heavily vegetated,
- Skew angle of flow to the bridge piers and abutments >0°,
- Shape and type of bridge piers
  - o are the bridge piers rectangular or hydraulically designed
  - o are the bridge piers simple or complex, meaning that they consist of more than one element which are lied down close to each other

4. **Is the flow velocity around the bridge during low and flood flows relatively high, small or negligible?**

In the case that based on the data from the previous steps a conclusion is made that bridge constricts the flow it is necessary to check if the flow velocities would be high or low during all possible conditions. If it is concluded that flow velocities would be negligible during all conditions (low and high flow and low and high tides in case that it is tidal at the bridge, previous conclusions on constriction to the flow would be reduced and new conclusion will be made that the constriction scour potential is low. Flow charts on Figure 11.10 and Figure 11.11 describe constriction evaluation procedure of inspector. All conclusions should be additionally elaborated and this should be documented as part of bridge scour inspection report.

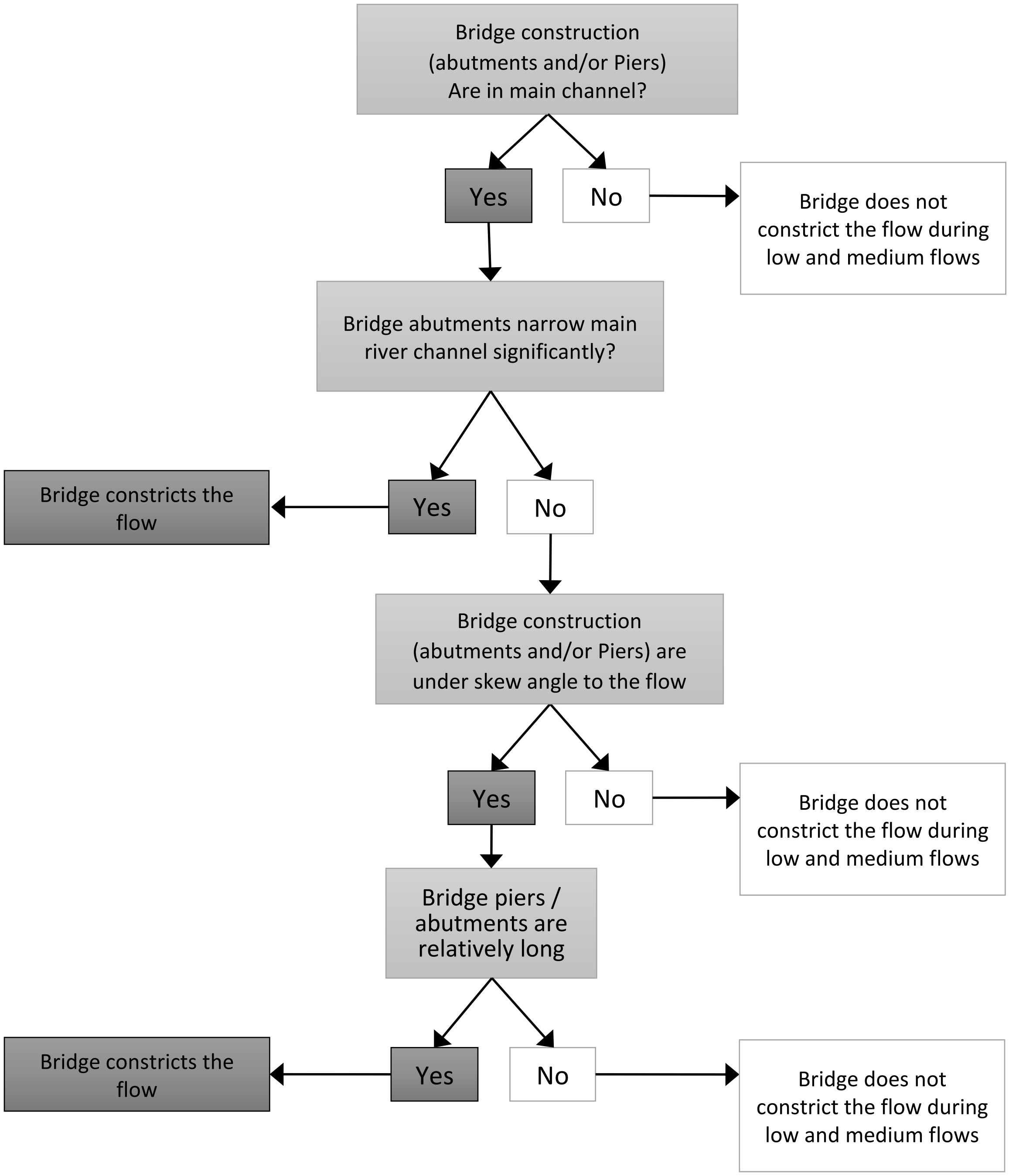


Figure 11.10. Methodology of constriction scour evaluation for low and mean flows.

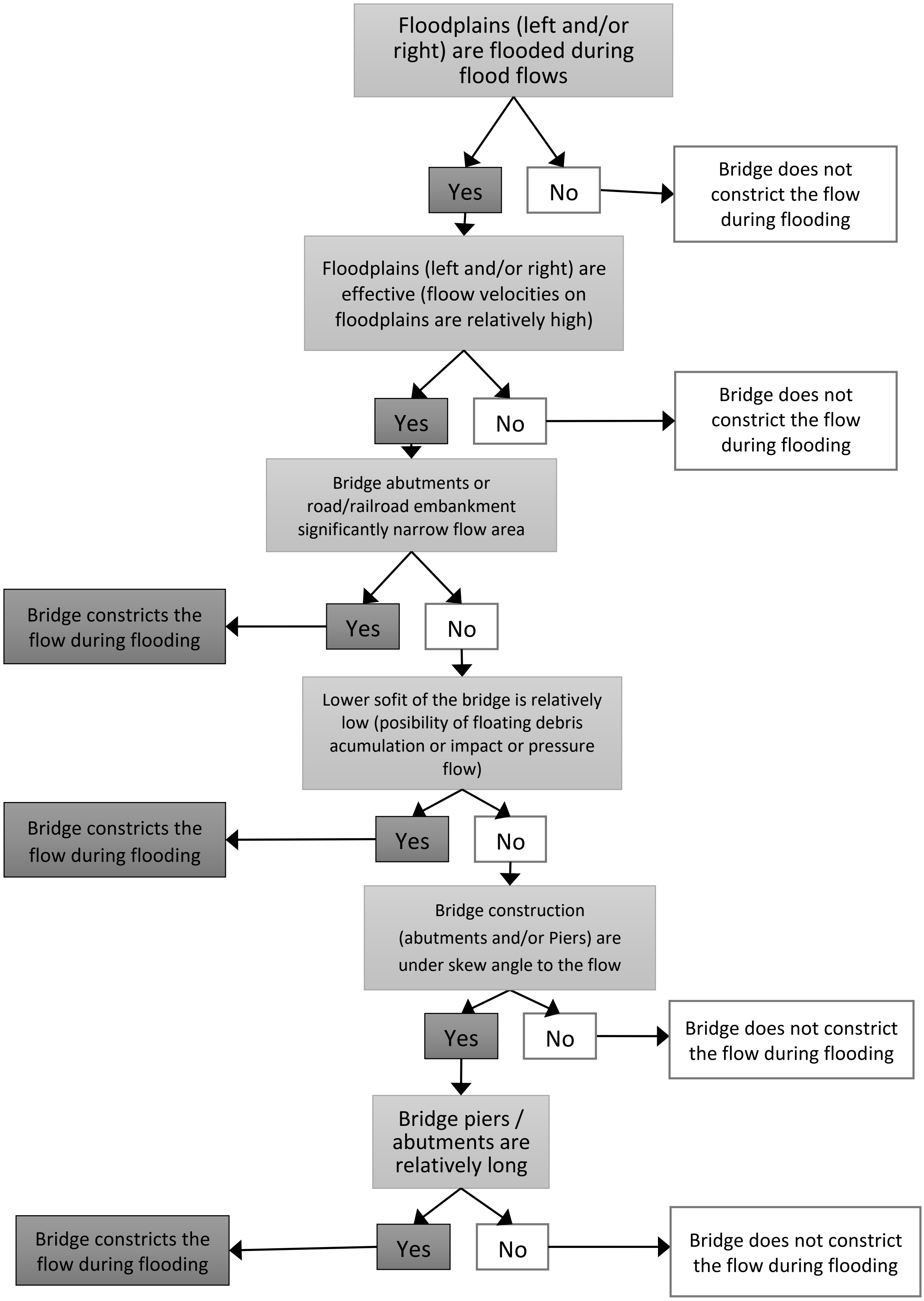


Figure 11.11. Methodology of constriction scour evaluation for flood flows.

I.8 Bridge site inspection

Bridge inspection is one of the most important steps in Stage 1. It consists on visual inspection of the bridge construction, riverbed material, main river channel (at the bridge and upstream and downstream of the bridge) riverbanks and floodplains. The inspection is documented in photographs and notes, voice recorders, etc. Some basic measurements of riverbed material, water depths or even simple methods of surface flow velocity measurements can be obtained during the inspections, but they wouldn't be mandatory to all of the bridges.

During the inspection it is necessary to take at least four photographs (two photographs of the bridge – one from upstream and one from downstream; and two photographs from the bridge – one looking upstream and one looking downstream). Additional photographs are possible based on the issues evident at the bridge (bridge piers/abutments, river banks, etc.).

Parameters to be considered during the inspection are:

- Possible undermining of bridge foundations
- Bridge stability in case of bridge scour around the bridge
- Lateral stability of the stream

During the bridge inspections three sections needs to be looked at. First section is area upstream of the bridge, second section is area around and at the bridge and the third section is area downstream of the bridge.

For the *section upstream of the bridge* it is necessary to observe and describe river banks (condition, material, vegetation, evidence of scour, etc.); observe main channel (riverbed material, width of the channel, depth of the channel, approach angle, weirs and or hydraulic structures in the channel); observe floodplains (vegetation, slope, objects or any other factors that could obstruct and change flow conditions at the floodplains during flooding).

For the *section at the bridge* it is necessary to evident details of bridge construction such as width and height of opening of the bridge, structure condition, scour around the bridge, evaluate of changes of the river bed when compared to previous inspections, condition of riverbed and or banks revetments (if any), condition of wing walls and installed scour protection (if any), note debris accumulation at the bridge, etc.

For the section downstream of the bridge it is necessary to evaluate condition of river banks, channel and floodplains (as for the upstream section).

Output data from the bridge scour inspection are described below.

I.8.1 General description

In this step it is necessary to lookup for bridge specifications. Bridge type needs to be determined. Based on the design of the bridge, the distinction will be made between a culvert and a bridge. Normally a culvert is a single span structure of width less than circa 2m.

Based on the hydraulic conditions at the bridge site, further distinction will be made between a "simple bridge" and a "complex bridge". A simple bridge needs to meet two criteria: it should be a single span bridge and it should be located on a uniform river section. In a case that one of two conditions is violated, then the bridge is termed as a "complex bridge".

Some of main bridge specifications are:

- Type of the bridge (culvert, simple bridge or complex bridge)
- Year of bridge construction
- Number of total bridge piers and spans
- Number of bridge piers in the river channel and spans over water
- Pier shape (are edges of the piers hydraulically designed)
- Are there any modifications on the bridge construction since its original design (describe changes if any)
- If possible, obtain drawings of the bridge (original and modifications)
- Are the foundations known or unknown?
- Is the bridge built on weir and are there any other factors that affect complexity of flow conditions at the bridge
- Other specific factors which describe bridge construction

I.8.2 Bed and bank material

Inspector needs to evaluate material of riverbeds and banks (clay, sand, gravel, boulders, rock etc.) and range of material grading. Also by applying a penetrating rod it is recommended to check material consistency and measure depth of penetration of the rood into the river bed.

I.8.3 Main channel

- It is necessary to determine if the bridge is located on relatively straight reach of the river ore in a curve which may pose skew angle of flow relative to bridge.
- Evaluate width of the main channel upstream of the bridge, at the bridge and downstream of the bridge.
- Define skew angle of the flow relative to bridge piers/abutments
- List of all structures upstream of the bridge, at the bridge and downstream of the bridge (natural dropdowns, increase of slope, weirs, sluices, etc.)
- Describe the range of water depths (riverbed / Thalweg elevation) upstream of the bridge, downstream of the bridge at the bridge. If there is no previous soundings of river bed it is recommended to obtainsoundings of the riverbed during the following inspection
- Compare existing cross sections with cross sections of previous surveys (if any) in order to determine if there is any degradation of river bed
- Comment existing longitudinal Thalweg profile and compare it with longitudinal riverbed profile from previous inspections (if any). Depressions in the riverbed upstream of the bridge, at the bridge and/or downstream of the bridge that could be indication of constriction scour.

I.8.4 River banks and embankments

It is necessary to describe bank condition (good, moderate, eroded, collapsed, etc.) and determine if there is vegetation on the banks. Trees on the banks could be an indication that the banks are stable. It is necessary to determine if river banks are natural or if they are embankments (man-made). If they riverbanks are embankments it is necessary to describe the type of embankments and condition of embankments (same as for riverbanks).

I.8.5 Bridge abutments

It is necessary to note if there are any scour countermeasures around the bridge abutments. If there are any, describe type and condition of scour countermeasure (good, deteriorated or collapsed). If data on foundation depths exists, it is necessary to provide drawings of abutment foundations and compare foundation depths to riverbed elevation. If there are any cracks of the bridge abutments it is required to estimate if this cracks could be a result of soil settlement under foundation or due to impact of the floating debris.

If conditions during the inspection are allowing it should be determined if there are any signs of undermining of foundations. This could be done by examining riverbed / water depths around the abutments with a rod/scale or even by taking underwater photograph. If this is not possible during the inspection, then analysis of undermining of foundations is obtained by inspecting cross sections of riverbed at upstream and downstream profile of the bridge and from plain view of riverbed soundings.

I.8.6 Bridge piers

Similar as for bridge abutments it is necessary to determine if there are any scour countermeasures around the bridge piers installed and evaluate their condition.

In case that there is no data, based on:

- pier type and material (masonry, concrete or steel)
- pier geometry (circular, rectangular, long or short)
- year construction

it can be determined type and approximate depth of foundations (caissons, pilots or possibly shallow foundations). When applying this methodology, the one should always be on safety side and if in doubt hallways assume shallow depths of foundations and recommend inspection of foundations if necessary.

In the case of steel piers which could be founded on caissons or pilots it is allowed to assume that foundation depth is equal to height of pier above waterline noted during the inspection (but not higher than that).

If data on foundation depths exists, it is necessary to provide drawings of pier foundations and compare foundation depths to riverbed elevation. If there are any cracks of the bridge piers it is required to estimate if this cracks could be a result of soil settlement under foundation or due to impact of the floating debris. Possible signs of undermining of pier foundations if possible should be inspected by using same principles as for bridge abutments.

I.9 Assessment of the bridge scour potential

I.9.1 General scour potential

Based on the gathered data, an expert hydraulic engineer provides overview of factors which influence or might influence on global scour. Based on the evidence or suspicion an expert gives rank on global scour potential around the bridge. It is required to specify if the global scour potential is related to the lateral and/or vertical instability. Ratings/ranks for general scour potential are showed in Table 11.21. Person which evaluates general scour potential (Table 11.21) in the third column records only the assigned rank.

Table 11.21. Modified BA74/06 ratings and ranks for general scour.

General scour potential	Rating	Rank
Channel stable upstream and downstream	1	
Channel unstable downstream	2	
Channel unstable upstream	3	
Channel unstable both upstream and downstream	4	

I.9.2 Constriction scour potential

Based on the gathered data, an expert hydraulic engineer provides overview of factors which influence or might influence on constriction scour. Based on the evidence or suspicion an expert gives rank on constriction scour potential around the bridge. Ratings/ranks for constriction scour potential are showed in Table 11.22. Person which evaluates constriction scour potential (Table 11.22) in the third column records only the assigned rank.

Table 11.22. Bekić-McKeogh Ratings and ranks for constriction scour.

Constriction scour potential	Rating	Rank
No constriction	1	
Constriction on one floodplain	2	
Constriction on both floodplains	3	
Significant alteration of natural flow conditions	4	

I.9.3 Local scour potential

In this step the main features of the bridge construction which influence on the local scour are described. In this step it is required to:

- Provide a list of areas with estimated scour extents and depths around the bridge which had evident scour during bridge inspection.
- Fill and answer questions from Table 11.23. Person which evaluates local scour writes and answers in the right column.

Table 11.23. Modified BA74/06 Features which affect local scour around the bridge.

Evidence to be looked for	*Observation*
Is bridge currently experiencing scour?	***Yes*** or ***No*** (if Yes, locate)
Has the bridge a history of scour problems as identified from inspection and maintenance records?	***Yes*** or ***No*** (if Yes, locate)
Are piers and abutments founded on shallow spread footings in the river channel?	***Known*** or ***Unknown*** (if known, describe type and depth)
Is the bridge on a steep river?	***Yes*** or ***No***
Is the bridge on an unstable river?	***Yes*** or ***No*** (specify if the river reach is laterally and/or vertically unstable)
Is the bridge on or immediately downstream of bends in the river?	***Yes*** or ***No***
Are piers subject to an oblique angle of attack from the flow?	***Yes*** or ***No*** (if yes, estimate skew angle)
Do abutments protrude into the river channel?	***Yes*** or ***No***
Are open spans of such lengths that the abutments or piers cause significant contraction of the river channel?	***Yes*** or ***No***
Is it a relatively small bridge opening or a bridge with obstructions that could be blocked by debris such as a tree: this could lead to increased velocities through the bridge opening and additional scouring of the bed?	***Yes*** or ***No***

I.9.4 Conclusions

This step consist of chart evaluation process shown in Figure 11.12. Figure 11.12 shows basic parameters and decisions needed to consider in Stage 1 – Assessment. Based on Figure 11.12 and conclusions from the steps 2.4.1 – 2.4.3 the bridge is evaluated with final rank based on which the bridge is assigned with the Priority rating from Table 11.19. If the evaluation process from Figure 2 suggests that Stage 2 is required, a conservative decision and recommendation to move to Stage 2 – Analysis is to be adopted. If the expert which assesses the bridge scour potential conclude that the Stage 2 – Analysis at that time is not necessary it is possible to evaluate the bridge scour potential as "Low risk", but with mandatory recommendation of next bridge inspection within 1 year.

I.9.5 Recommendations

Beside the assignment of Priority rating, number of years to next inspection and Rank Summary this is the most important part of the method. Person qualified for providing recommendations needs to be of the hydraulic and/or the geologic education.

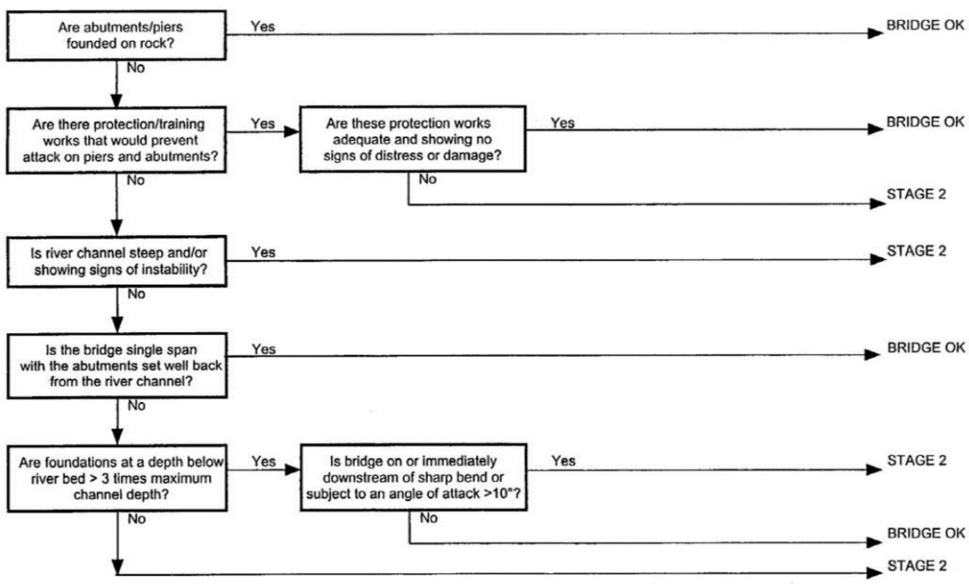


Figure 11.12. Basic parameters and decisions needed to be assessed in Stage 1 [127].

Annex L Detailed pair-wise comparison of methods B, C and L

L.1 Comparison results for Data block 1

The comparison is made for 44 single span railway bridges across Ireland. The main purpose of this comparison is to validate Method L1 with Method B1. Full comparison is made between Priority Rating (PR) obtained by Method B1 (Modified BA74/06 Bekić-McKeogh), Method L1 Scour Condition Rating (L1.ScCR), Method L2 Scour Condition Rating (L2.ScCR) from the newly developed Scour Inspection Module and Method C Categories. Table 11.52 below shows preferable (expected) outcomes when Method B1, Method L1 and L2 and Method C are applied on the same bridge. This means that if the result of the bridge inspection is Priority Rating PR = 1 (in the case when Method B1 is applied), then the expected result for applying Method L1 (Scour Inspection for Level 1 Bridges) or Method L2 (Scour Inspection for Level 2 Bridges) should be Level 1 Scour Condition Rating of L1.ScCR = 0 (No or insignificant damage) and Level 2 Scour Condition Rating of L2.ScCR = 0 (No or insignificant damage) respectively. The Methods L1 and L2 have refined the Priority Rating PR 2 "Low Risk" into two Condition Ratings L1(L2).ScCR 1 "Minor damage but no need of repair" and L1(L2).ScCR 2 "Some damage, repair needed when convenient". This finer refinement of Scour Condition Rating (when compared with Priority Rating PR 2) makes a more appropriate distinction between bridges with potential scour risk (L1(L2).ScCR 1) and bridges where there is an evidence of scour risk (L1(L2).ScCR 2) which could be mitigated with some minor repair works. By applying both methods B1 and L1 the anticipated outcome is to move to the Stage 2 Analysis (Method B1) or Proceed to Level 2 inspection (Method L1). For the same case in Method L2 an acceptable Scour Condition Ratings would be L2.ScCR 3 (Significant damage) or L2.ScCR 4 (Damage is critical), in accordance with Table 11.52 below.

Table 11.52. DB 1 - Matrix showing when the results of scour inspections are comparable

	Expected results from bridge inspections for the same bridge Priority rating / L1.ScCR and L2.ScCR				
Method B1 **Priority Rating (PR)**	1 Insignificant Risk.	2 Low risk (maintenance, minor actions).		3 Move to Stage 2 - Analysis.	
Method L1 **Scour Condition Rating (L1.ScCR)**	0 No or insignificant damage.	1 Minor damage but no need of repair.	2 Some damage, repair needed when convenient.	3[44] Proceed to Level 2 inspection.	
Method L2 **Scour Condition Rating (L2.ScCR)**	0 No or insignificant damage.	1 Minor damage but no need of repair.	2 Some damage, repair needed when convenient.	3 Significant damage, repair needed within next financial year.	4 Damage is critical. It is necessary to execute repair works or scour risk management at once
Method C **(Category)**	6 Low Priority	5 Low Priority	4 Medium Priority	3 Medium Priority	2 High Priority

[44] In Level 1 Bridge Scour Inspection, Scour Condition Rating ScCR 3 is not assigned, yet the bridge is recommended to Proceed to Level 2 Scour Inspection

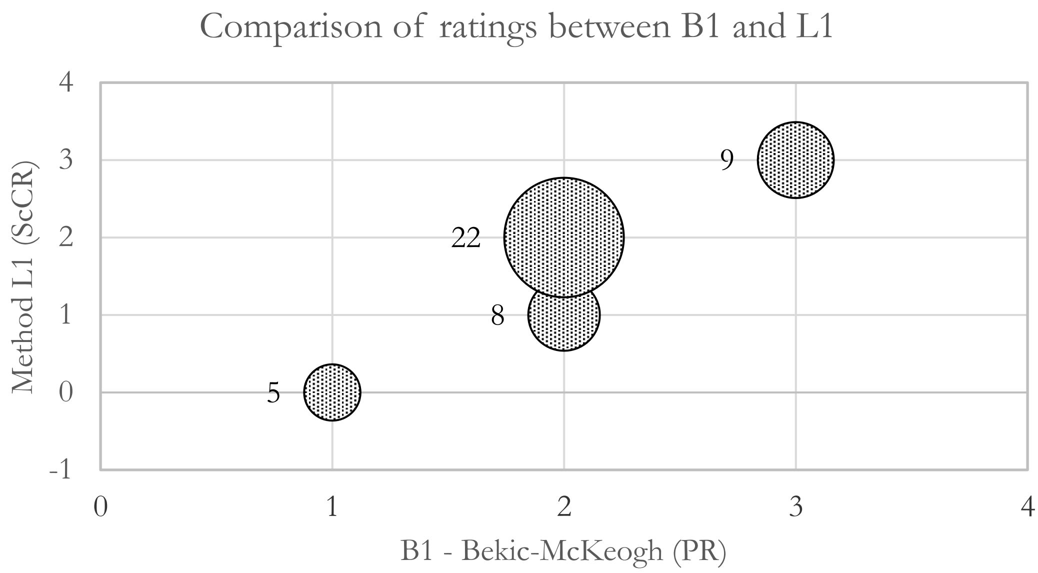
L.1.1 Comparison results between Method B1 and L1

The results (Figure 11.69) indicate that correlation ($R^2 = 0.82$) between Method B1 and Method L1 (L1.ScCR) is strong [162]. Note that the bubble size in Figure 11.69 presents the number and percentage of the bridges respectively. For five bridges (11.4%) that had PR 1 (insignificant risk) from Method B1, Method L1 assigned Scour Condition Rating L1.ScCR 0 (No or insignificant damage). For bridges that Method B1 gained PR 2 (Low risk), Method L1 assigned Scour Condition Rating L1.ScCR 1 (Minor damage) for eight bridges (18.2%) and L1.ScCR 2 (Some damage) for 22 bridges (50.0%). For nine bridges (20.5%) that had PR 3 (Move to Stage 2 – Analysis), Method L1 recommended to Proceed to Level 2 inspection (L2). No discrepancies from the preferred results are noted (see Table 11.52).

During the inspection procedure it was noted that for one bridge (2.3%), Method L1 could potentially underestimate the bridge Scour Condition Rating. This is noted for the bridge named "UB154" over Craughwell River on the Limerick/Tuam railway line. If the inspector's decision was to opt for state C, the bridge Condition Rating of L1.ScCR = 2 would be lower than Method B1 PR = 3 due to a marginal decision for a component 8 (L1.Sc.c8). The recorded scour depth is between 0.5m and 0.6m, implying the inspector needs to decide between state C ("Scour depth <0.6m or sedimentation present. Bank erosion <1.0m to bridge abutments") and state D (Scour depth >0.6m or undermining of bridge abutments, go to Level 2). The final decision was to opt state D, so in the final results there is no difference between the ratings of Method B1 and Method L1. However, this implies that last four components (L1.Sc.c8, L1.Sc.c9, L1.Sc.c10 and L1.Sc.c11) have more significance to the final results than the previous seven components (L1.Sc.c1 to L1.Sc.c7).

The overall conclusion from the comparison is that zero bridges, e.g. (0%) of the results using Method L1 have unacceptable results.

If we observe the recommended years to next inspection, the results indicate much more variations between bridges (Figure 11.70) when compared to recommended years to next inspection for Method B1. In method L1, most of the results fall into category 0 years, 4 years and 6 years (in accordance with Table 6.6-Table 6.7). This difference is as the criteria for years to next inspections in Method L (L1 and L2) have been significantly altered following interviews with bridge managers in Ireland and Portugal. It should be noted that 0 years to next inspection means “as soon as possible within the same financial year”.



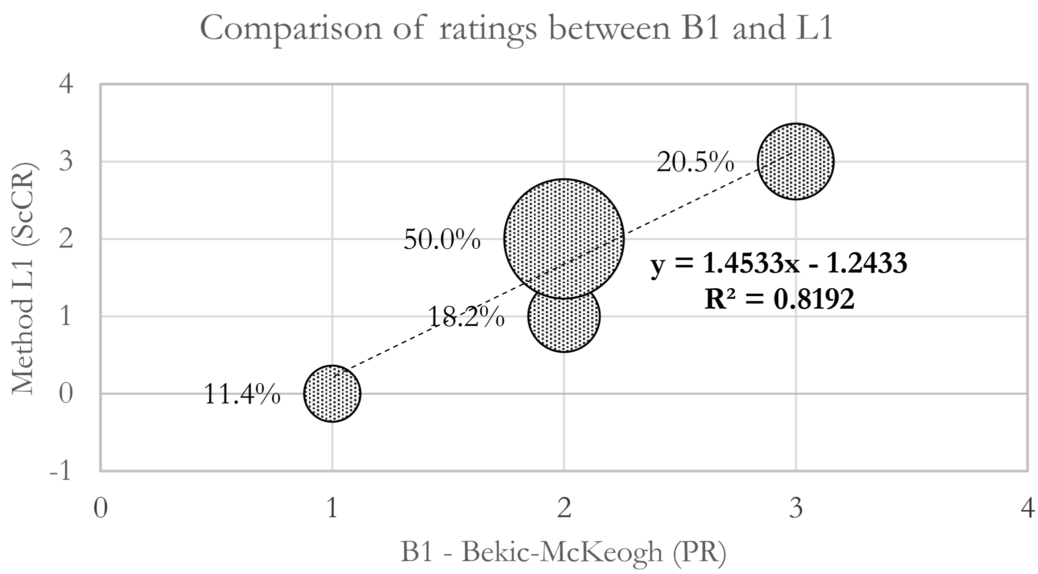


Figure 11.69 Results of the comparison between Method B1 and L1 for Data block 1.

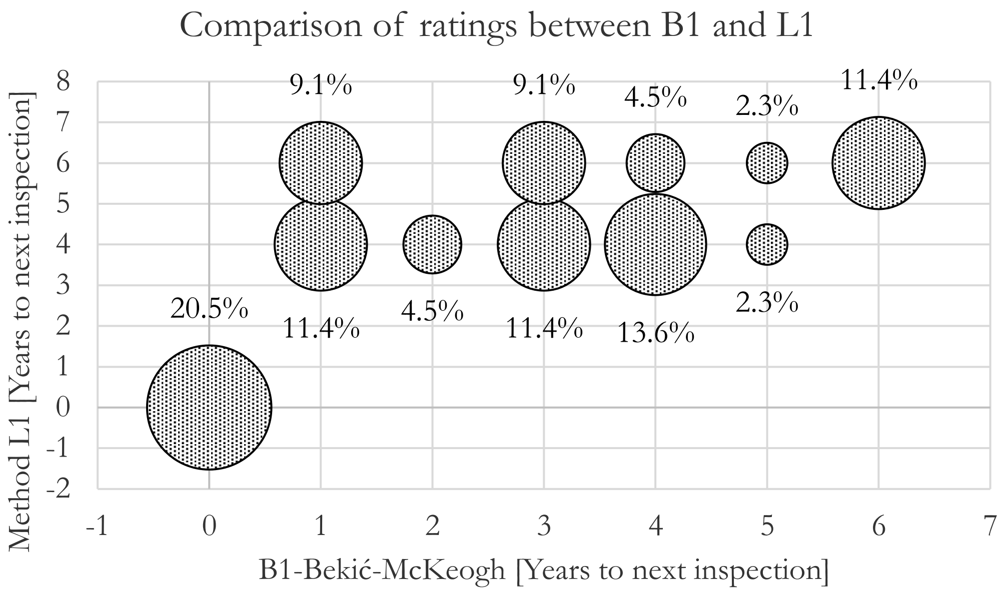
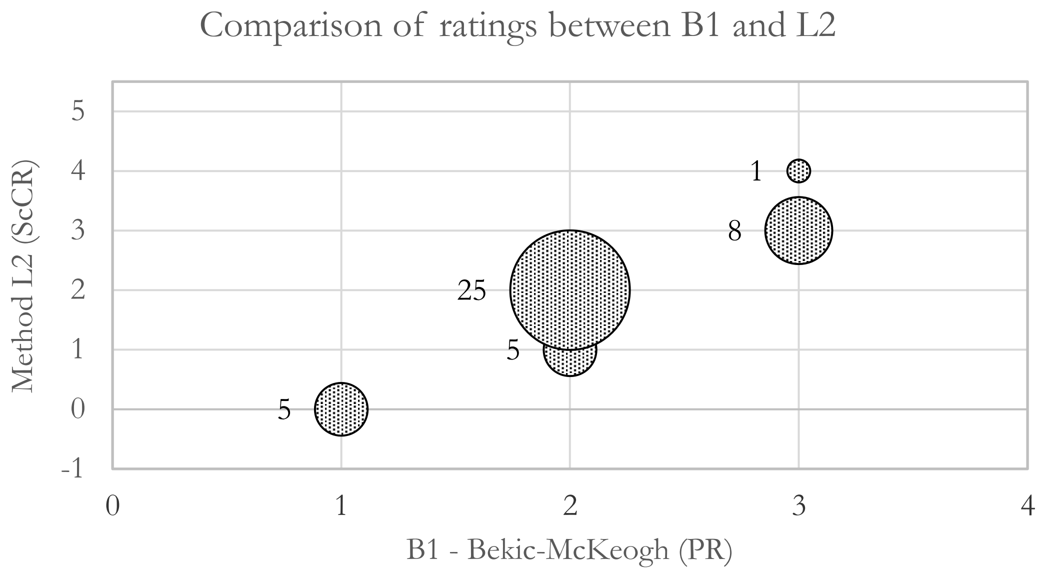


Figure 11.70. Comparison of recommended years to next inspection between Method B1 and L1.

L.1.2 Comparison results between Method B1 and L2

The results (Figure 11.71) indicate that correlation ($R^2 = 0.84$) between Method B1 and Method L2 (L1.ScCR) is strong [162]. Note that the bubble size in Figure 11.71 presents the number and percentage of the bridges respectively. The same as for Method L1, for five bridges (11.4%) that had PR 1 (insignificant risk) from Method B1, Method L2 assigned Scour Condition Rating L1.ScCR 0 (No or insignificant damage). For bridges that Method B1 gained PR 2 (Low risk), Method L2 assigned Scour Condition Rating L2.ScCR 1 (Minor damage) for five bridges (11.4%) and L2.ScCR 2 (Some damage) for 25 bridges (56.8%). For nine bridges (20.5%) that had PR 3 (Move to Stage 2 – Analysis), Method L2 assigned eight bridges (18.2%) Scour Condition Rating L2.ScCR 3 (Significant damage) and one bridge (2.3%) Scour Condition Rating L2.ScCR 4 (Damage is critical). No discrepancies from the preferred results are noted (see Table 11.52). Some differences for four bridges (9.1%) are noted when compared with Level 1 (Method L1). This will be more closely looked in the following section L.1.4.



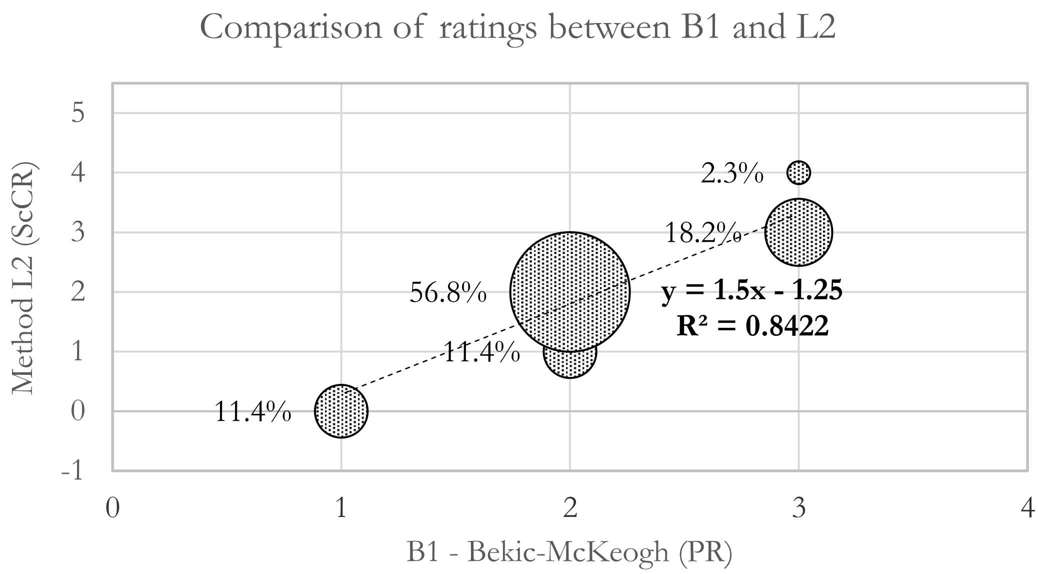
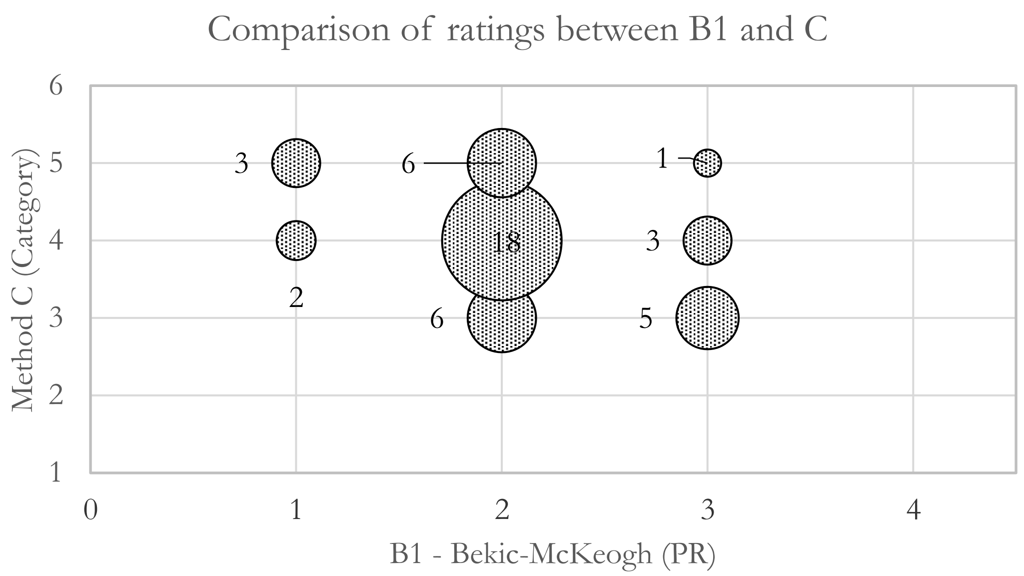


Figure 11.71 Results of the comparison between Method B1 and L2 for Data block 1.

L.1.3 Comparison results between Method B1 and C

The results (Figure 11.72) indicate that correlation (R^2 = 0.17) between Method B1 and Method C is very weak [162]. For expected results refer to Table 11.52. Five (5) bridges that are ranked with PR = 1 using Method B1 have expected Category of 6. However, Method C ranked this bridges with category 5 and category 4 respectively. Furthermore, for six (6) bridges Method B1 gained PR = 2 while Method C categorises them with category 5 and 4, instead with category 3 or two. The overall conclusion from the comparison is that for fifteen (15) bridges, e.g. (34%) results using Method C are unreliable.



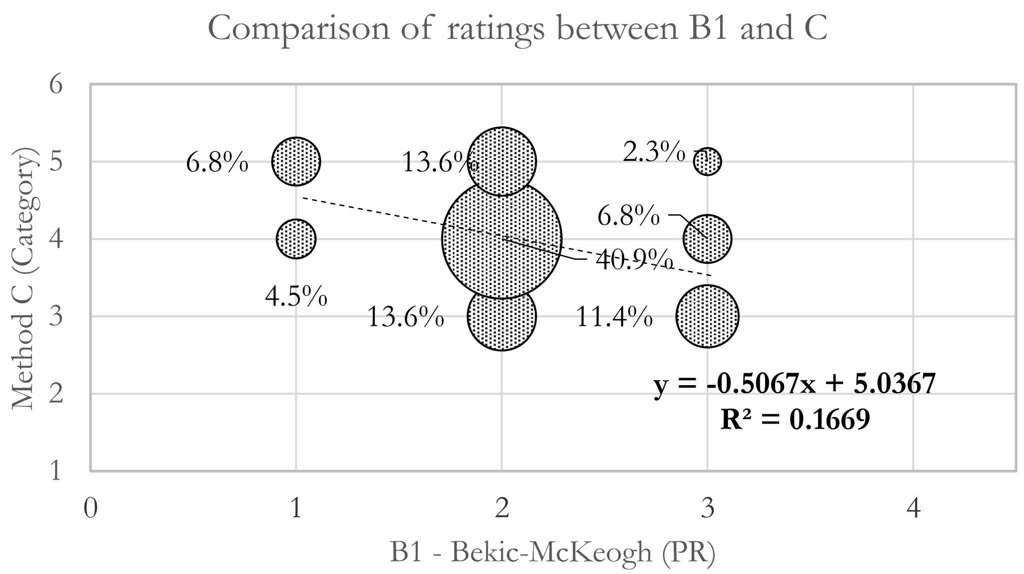


Figure 11.72 Results of the comparison between Method B1 and C for Data block 1.

L.1.4 Comparison results between Method L1 and L2

The results (Figure 11.73) suggest strong correlation ($R^2 = 0.75$) between Method L1 and Method L2 [162].

The noted differences between the Scour Condition Ratings for Method L1 (Figure 11.69) and Method L2 (Figure 11.71) will be assessed in more detail in this section. Note that the bubble size in figures below presents the number and percentage of the bridges respectively.

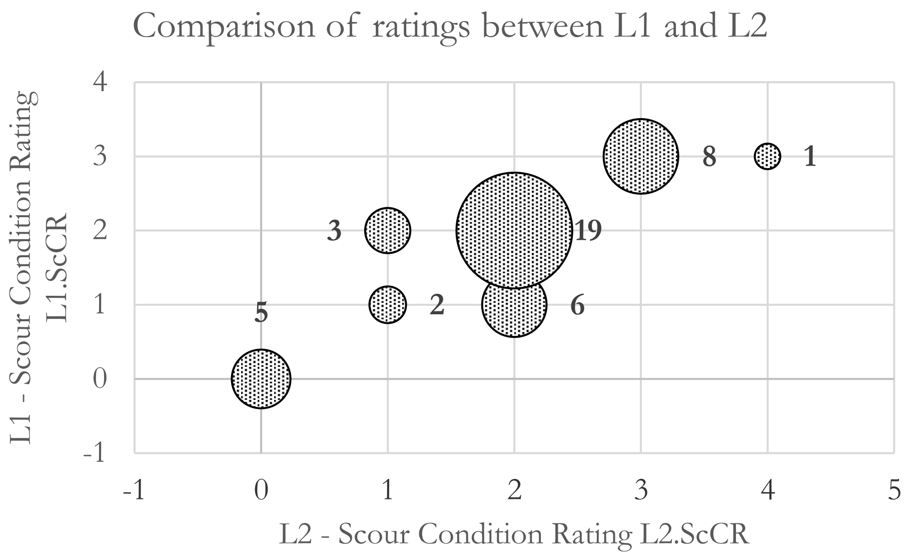
Level 2 assessment gave one level higher Condition rating for four bridges (9.1%) when compared with Level 1 (Method L1).

For three bridges (6.8%) that were evaluated with L1.ScCR = 1 (Minor Damage) in Level 1 assessment, Level 2 assessment gained Scour Condition Rating ScCR = 2 (Some damage). Further, out of nine bridges that L1 assessment recommended to "Proceed to Level 2 inspection", Level 2 inspection assigned eight bridges (18.2%) Scour Condition Rating L2.ScCR 3 (Significant damage) and one bridge (2.3%) Scour Condition Rating L2.ScCR 4 (Damage is critical). Meaning that one bridge (2.3%) gave larger rating. All this differences are acceptable and, according to Table 11.52 are considered reliable.

This more detailed analysis with comments above confirmed that correlation ($R^2 = 0.75$) between Method L1 and Method L2 (L1.ScCR) remains strong [162].

When comparing the Years (Time) to next inspection between Method L1 and L2, more significant difference can be noted (Figure 11.74). The differences are apparent for eight bridges (18.2%) that from Method L1 are recommended to "Proceed to Level 2 inspection" as soon as possible. This is not considered as major flaw of Method L1 as it only suggests that more detailed inspection is required as soon as possible.

The only remaining difference is apparent for nine bridges (20.5%), where Level 2 recommended to conduct next inspection within next 4 years and Level 1 inspection recommended re-inspection for the same bridge within next 6 years. This difference would not have major consequence on the condition of the bridge or its maintenance. It simply gives more conservative recommendation in case the bridge is inspected with simplified inspection – Method L1.



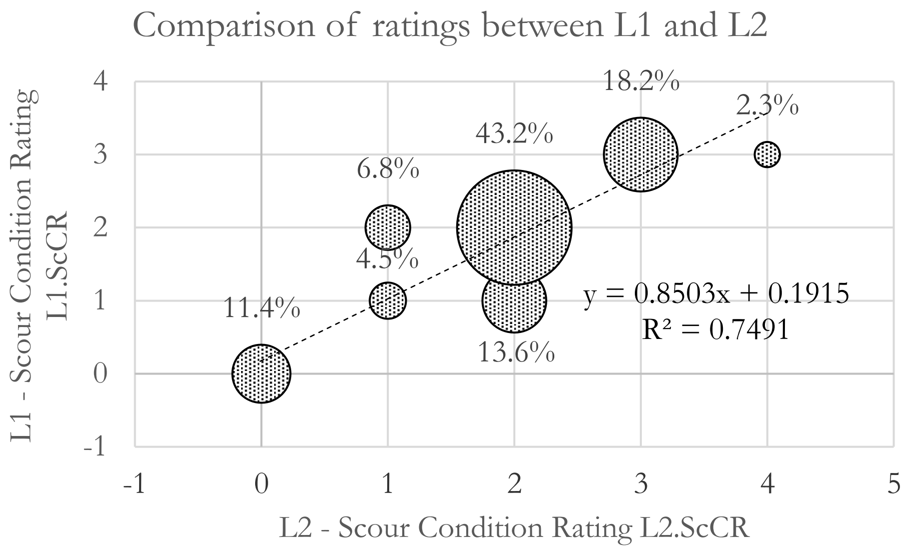


Figure 11.73 Results of the comparison between Method L1 and L2 for Data block 1.

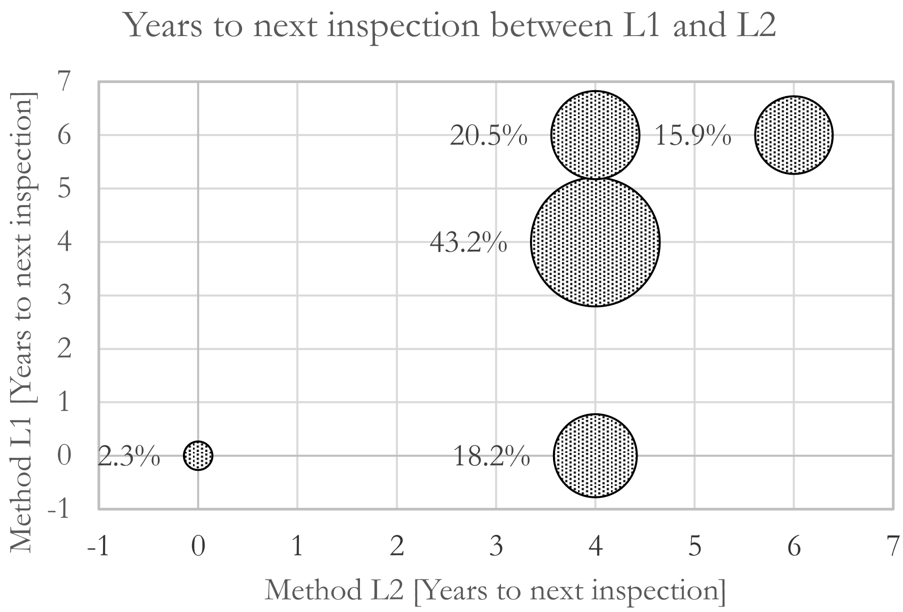
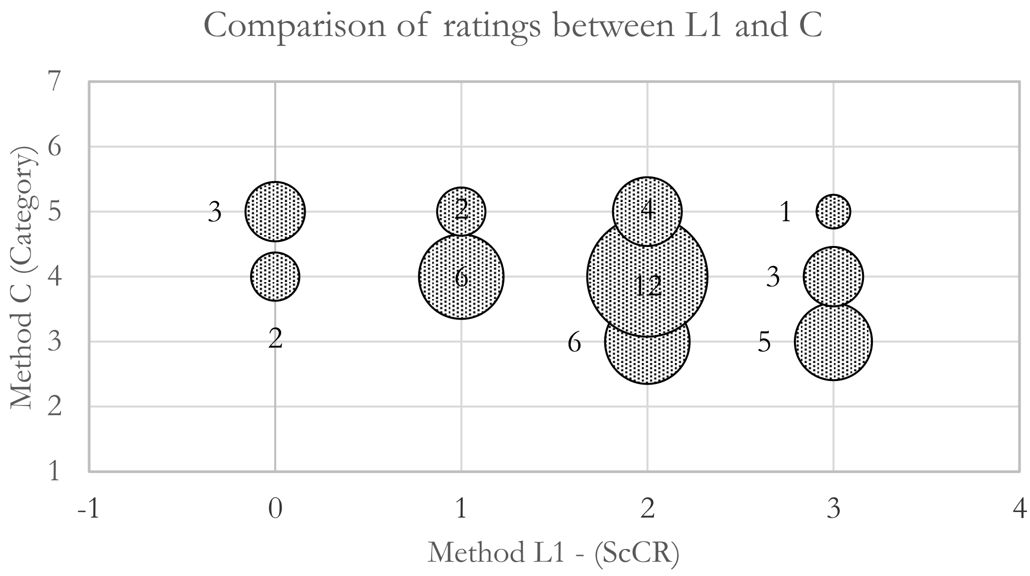


Figure 11.74. Comparison of recommended years to next inspection between Method L1 and L2.

L.1.5 Comparison results between method L1 and C

The scour condition rating L1.ScCR from the new inspection module, e.g. Method L1 is compared with Handbook 47 Method C categories. The comparison shows the same trend, however the results indicate that correlation (R^2 = 0.202) between Method L1 (L1.ScCR) and Method C categories is very weak [162]. It should be noted that Categories in Method C are dispersed relative to their Priorities. The Category 1 and 2 have High Priority, Category 3-4 have medium Priority and Category 5-6 have Low priority, see Table 4.10. if we observe the priorites from the Method C, all bridges are ranked within priorities High and Medium. No bridge is ranked as Low Priority according the Method C, which indicates that this method is surely not completely adequate or that Priorities in Method C need to be updated. Comparisson between Method B1 (basis for the comparison) and Method C already suggested that Method C might be unreliable, see section L.1.3

In the following section, further comparison will be made on a larger sample of 101 bridges.



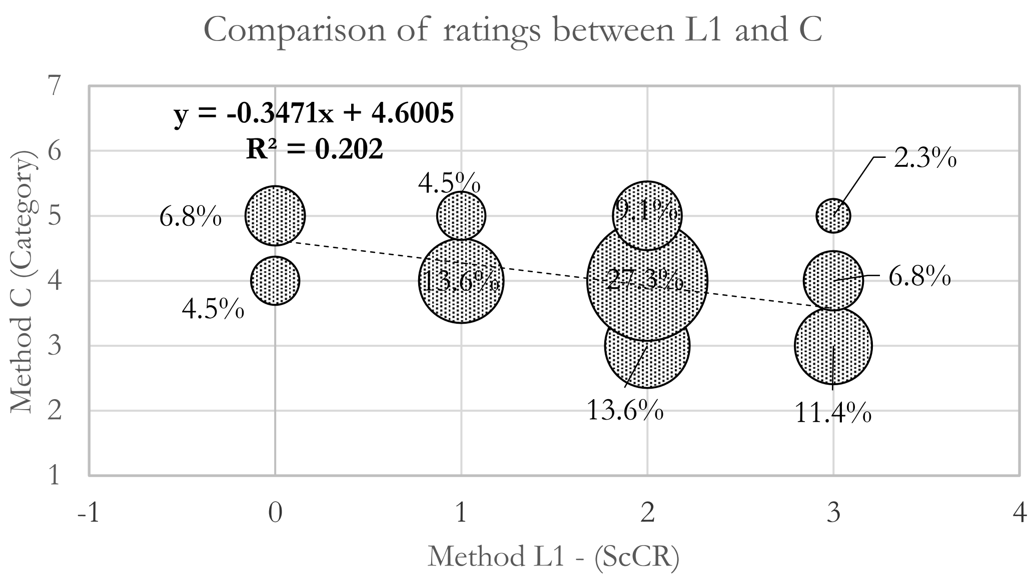


Figure 11.75 Results of the comparison between Method L1 and C for Data block 1.

L.2 Comparison results for Data block 2

The comparison is made for 101 railway bridges across Ireland. The main purpose of this comparison is to validate Method L2 with Method B1. The comparison is made between Priority Rating (PR) obtained by Method B1 (Modified BA74/06 Bekić-McKeogh), Method L2 Scour Condition Rating (L2.ScCR) from the newly developed Scour Inspection Module and Method C Categories.

A full comparison between Method B1 and L2 was made for dataset of 101 railway bridges across the Ireland. The comparison is made between Priority Rating (PR) obtained by Method B1 (Bekić-McKeogh) and Method L2 Scour Condition Rating (L2.ScCR) from the newly developed Scour Inspection Module. Table 11.53 below shows anticipated outcomes when Method B1, L2 and C are applied on the same bridge. This means that if the result of the bridge inspection is Priority Rating PR = 1 (in case when Method B1 is applied), then the expected result for applying the Method L2 (Scour Inspection for Level 2 Bridges) should be Level 2 Scour Condition Rating of L2.ScCR = 0 (No or insignificant damage). The Method L2 has refined the Priority Rating PR 2 "Low Risk" into two Condition Ratings L2.ScCR 1 "Minor damage but no need of repair" and L2.ScCR 2 "Some damage, repair needed when convenient".

For PR 3 (Move to Stage 2 Analysis) from Method B1, acceptable Scour Condition Ratings in Method L2 would be L2.ScCR 3 (Significant damage) or L2.ScCR 4 (Damage is critical). Lastly, for PR 4 (Immediate action required) in Method B1, acceptable Scour Condition Ratings in Method L2 would be L2.ScCR 4 (Damage is critical) or L2.ScCR 5 (Ultimate damage), in accordance with Table 11.53 below.

Table 11.53. DB 2 - Matrix showing when the results of scour inspections are comparable

	Expected results from bridge inspections for the same bridge Priority rating / L2.ScCR					
Method B1 Priority Rating (PR)	1 Insignificant Risk.	2 Low risk (maintenance, minor actions).		3 Move to Stage 2 - Analysis.		4 Immediate action required (PoA).
Method L2 Scour Condition Rating (L2.ScCR)	0 No or insignificant damage.	1 Minor damage but no need of repair.	2 Some damage, repair needed when convenient.	3 Significant damage, repair needed within next financial year.	4 Damage is critical. It is necessary to execute repair works or scour risk management at once	5 Ultimate damage. The component has failed or is in danger of total failure
Method C Categories	6 Low Priority	5 Low Priority	4 Medium Priority	3 Medium Priority	2 High Priority	1 High Priority

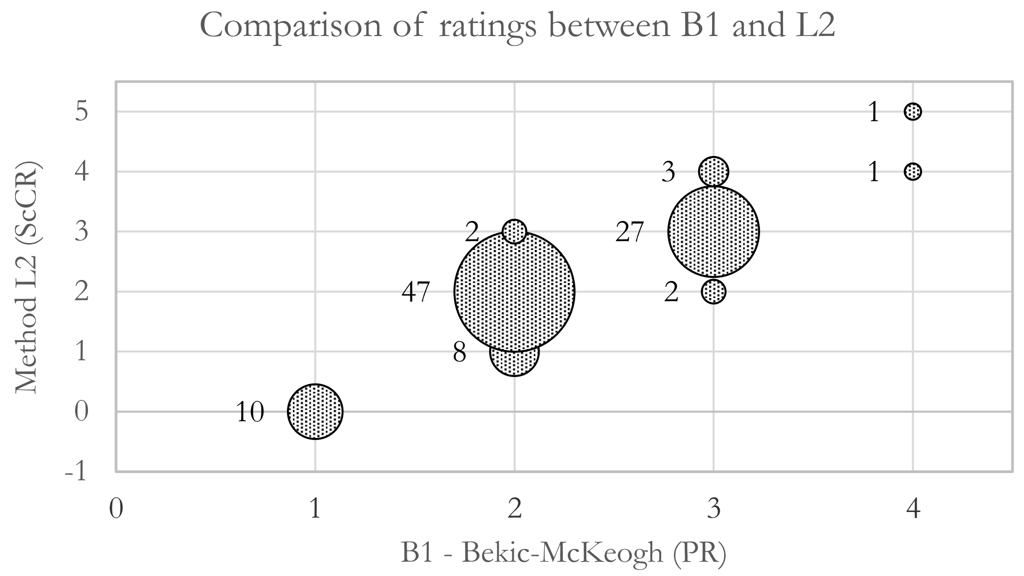
L.2.1 Comparison results between Method B1 and L2

The results (Figure 11.76) indicate that correlation ($R^2 = 0.82$) between Method B1 and Method L1 (L1.ScCR) is strong [162]. Note that the bubble size in figures below presents the number and percentage of the bridges respectively. For ten bridges (9.9%) that gained Priority Rating PR 1 (Insignificant risk) from Method B1, Method L2 evaluated all of those bridges with Scour Condition Rating L2.ScCR 0 (No or insignificant damage). For fifty-seven bridges (56.4%) that gained Priority Rating 2 (Low risk) from Method B1, Method L2 evaluated eight bridges (7.9%) with L2.ScCR 1 (Minor damage) and forty-seven bridges (46.5%) with L2.ScCR 2 (Some damage). For two bridges (2.0%), Level 2 assigned one level higher Scour Condition Rating L2.ScCR 3 (Significant damage). This will be looked at in more detail.

For thirty-two bridges (31.7%) that gained Priority Rating 3 (Move to Stage 2 Analysis) from Method B1, Method L2 evaluated twenty-seven bridges (26.7%) with L2.ScCR 3 (Significant damage) and three bridges (3.0%) with L2.ScCR 4 (Damage is critical). For two bridges (2.0%), Level 2 assigned one lower Scour Condition Rating L2.ScCR 2 (Some damage). This will be looked at in more detail.

The overall conclusion from the comparison is that for four bridges, e.g. (4%) the results using Method L2 gave one level higher or lower Scour Condition Rating than anticipated.

If we observe the recommendations for years to next inspection between Method B1 and L2 results indicate much more variations (Figure 11.77). Similarly as for Level 1 inspections, most of the results in Method L2 fall down into category 0 years, 4 years and 6 years (in accordance with Table 6.7. This difference is as the criteria for years to next inspections in Method L1 and L2 have been significantly altered after conducted interviews with bridge managers in Ireland and Portugal.



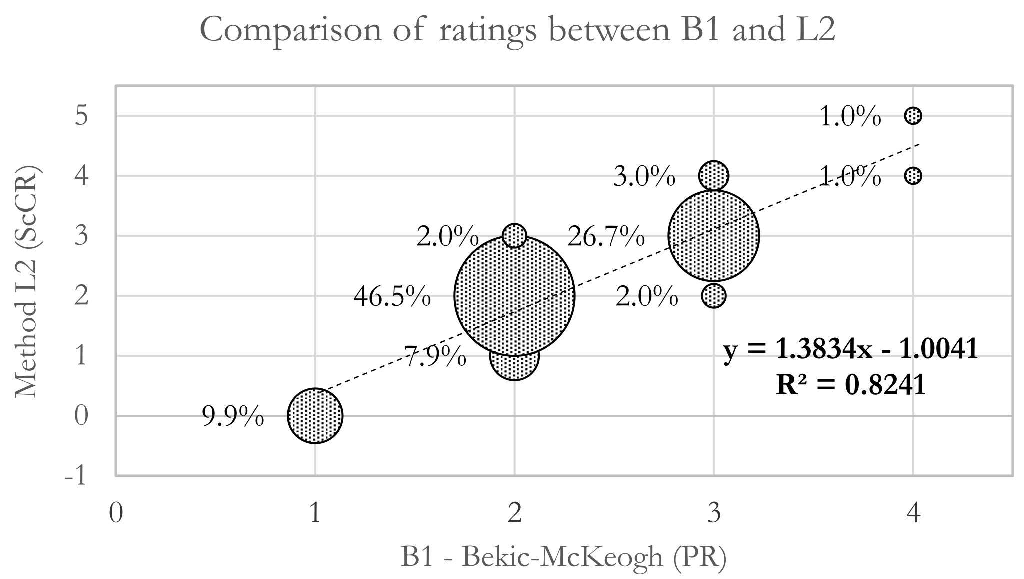


Figure 11.76 Results of the comparison between Method B1 and L2 for Data block 2.

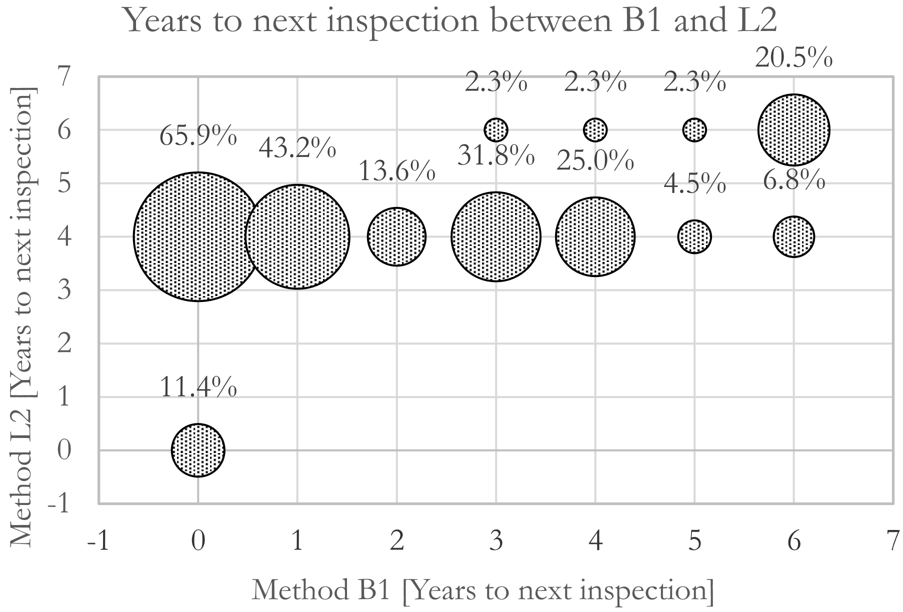
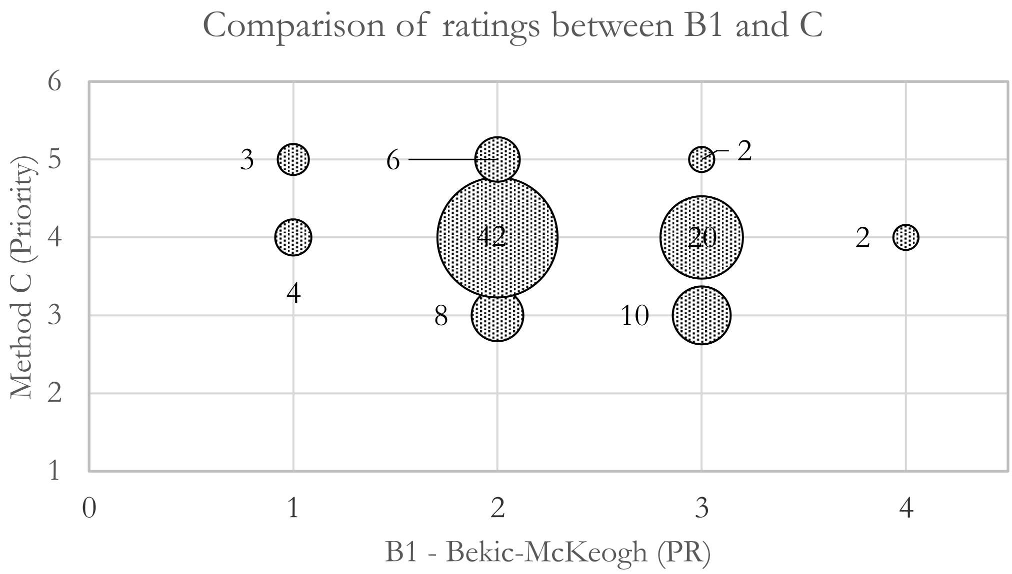


Figure 11.77. Comparison of recommended years to next inspection between Method B1 and L2.

L.2.2 Comparison results between Method B1 and C

The results (Figure 11.78) indicate that correlation ($R^2 = 0.15$) between Method B1 and Method C is very weak [162]. For expected results refer to Table 11.53. Seven (7) bridges that are ranked with PR = 1 using Method B1 have expected Category of 6. However, Method C ranked this bridges with category 5 and category 4 respectively. Furthermore, for eight (8) bridges Method B1 gained PR = 2 while Method C categorises them with with category 3, instead with category 5 and 4. Twenty-two (22) bridges ranked with Method B1 PR = 3 have categories of 5 and 4, instead category 3 or 2. Lastly, two bridges ranked with Method B1 as PR = 4 “Immediate Action Required”, while Method C assigns category 4. The overall conclusion from the comparison is that for thirty-nine (39) bridges, e.g. (39%) the results using Method C are unreliable.



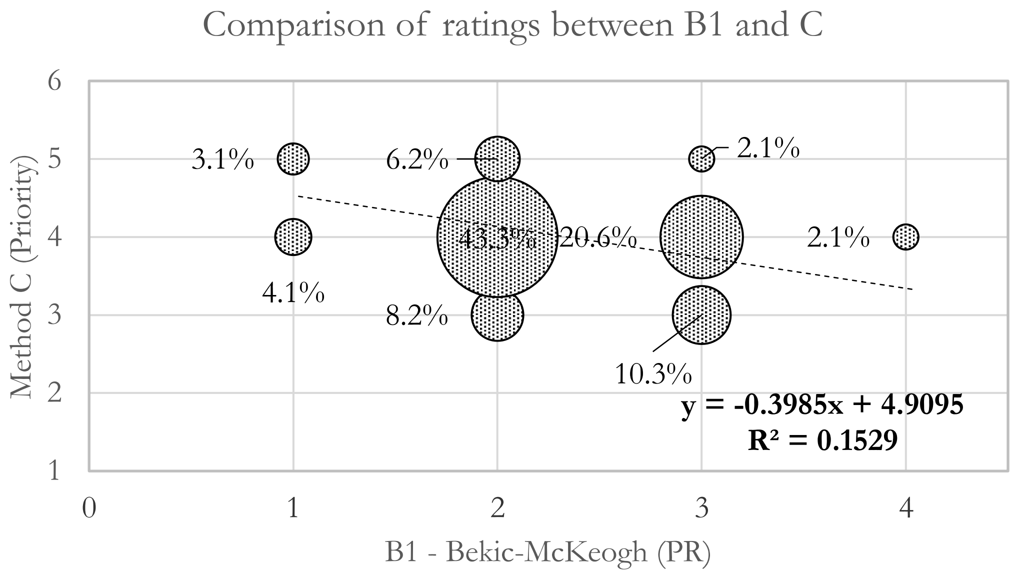
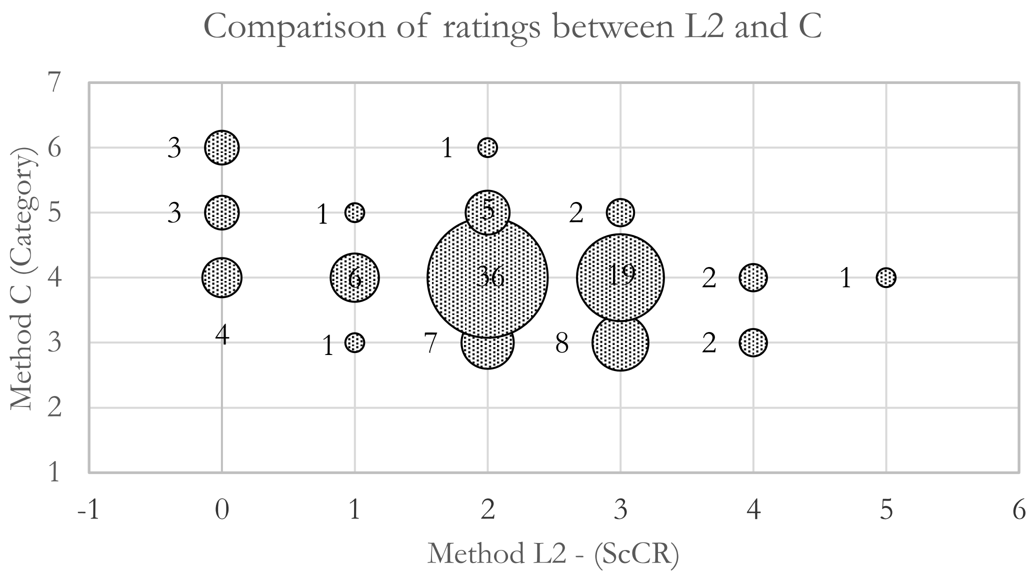


Figure 11.78 Results of the comparison between Method B1 and C for Data block 2.

L.2.3 Comparison results between method L2 and C

The scour condition rating L2.ScCR from the new inspection module for detailed bridge scour inspection (L2) is compared with Handbook 47 method (C) categories. The comparison shows the same trend, however the results indicate that correlation (R^2 = 0.17) between Method L1 (L1.ScCR) and method C Category is very weak [162]. It should be noted that Categories in Method C are dispersed relative to their Priorities, see Table 4.10. The same as when comparing to Method L1, if we observe the priorites from the

Method C, all bridges are ranked within priorities High and Medium. No bridge is ranked as Low Priority according the Method C, which indicates that this method is surely not completely adequate or that Priorities in Method C need to be updated.



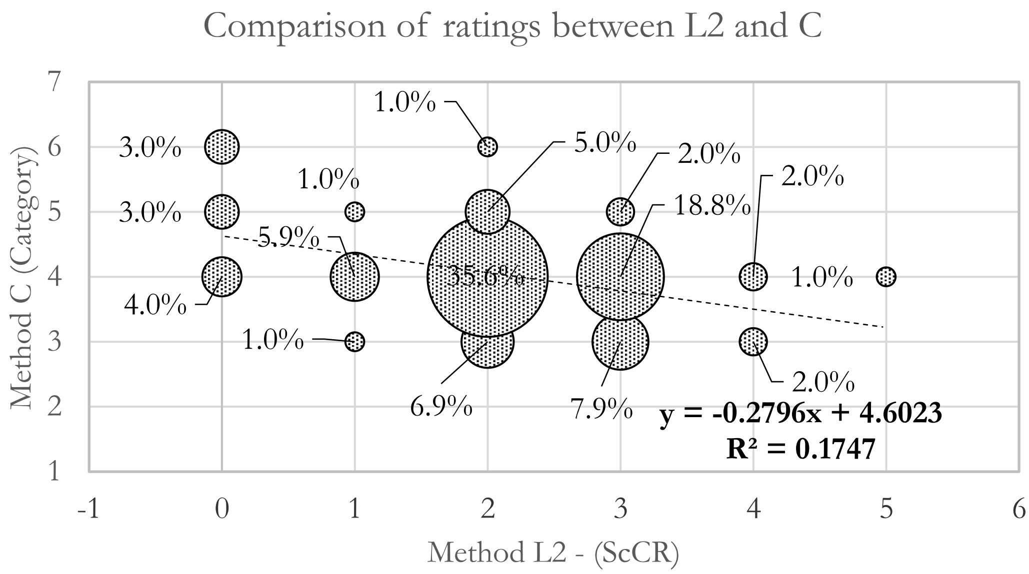


Figure 11.79 Results of the comparison between Method L2 and C for Data block 2.

L.3 Comparison of results for Data block 3

Data block 3 consists of 25 railway bridges in Ireland to which Methods B2a and B2a were applied. The main purpose of this comparison is to validate Method B2a and B2b and to compare their ratings with Method L1 and L2. The comparison is made between Priority Rating (PR) obtained by Method B1 (Modified BA74/06 Bekić-McKeogh), Method L1 Scour Condition Rating (L1.ScCR), Method L2 Scour Condition Rating (L2.ScCR) from the newly developed Scour Inspection Module, Method B2a Scour Risk Rating (SRR), Method B2b Qualitative Rating (QR) and Method C Categories. Table 11.54 below shows anticipated outcomes when Methods B1, L1, L2, B2a, B2b and C are applied on the same bridge.

Table 11.54. DB 3 - Matrix showing when the results of scour inspections are comparable

<table>
<tr><td></td><td colspan="7">Expected results from bridge inspections for the same bridge Priority rating / L2.ScCR</td></tr>
<tr><td>Method B1 Priority Rating (PR)</td><td>1
Insignificant Risk.</td><td colspan="2">2
Low risk
(maintenance, minor actions).</td><td colspan="2">3
Move to Stage 2 - Analysis.</td><td colspan="2">4
Immediate action required (PoA).</td></tr>
<tr><td>Method L1 Scour Condition Rating (L1.ScCR)</td><td>0
No or insignificant damage.</td><td>1
Minor damage but no need of repair.</td><td>2
Some damage, repair needed when convenient.</td><td colspan="4">3[45]
Proceed to Level 2 inspection.</td></tr>
<tr><td>Method L2 Scour Condition Rating (L2.ScCR)</td><td>0
No or insignificant damage.</td><td>1
Minor damage but no need of repair.</td><td>2
Some damage, repair needed when convenient.</td><td>3
Significant damage, repair needed within next financial year.</td><td colspan="2">4
Damage is critical. It is necessary to execute repair works or scour risk management at once</td><td>5
Ultimate damage. The component has failed or is in danger of total failure</td></tr>
<tr><td>Method B2a (SRR)</td><td>5
No action required</td><td colspan="2">4-3
Further investigations, Re-inspections</td><td colspan="2">2
Determine need for monitoring and scour protection measures as a high priority</td><td colspan="2">1
Immediate Risk Structures</td></tr>
<tr><td>Method B2b (QR)</td><td>1-4
Negligible risk</td><td colspan="2">5-9
Tolerable risk</td><td colspan="2">10-15
Undesirable risk</td><td colspan="2">>15
Intolerable</td></tr>
<tr><td>Method C Categories</td><td>6
Low Priority</td><td>5
Low Priority</td><td>4
Medium Priority</td><td>3
Medium Priority</td><td colspan="2">2
High Priority</td><td>1
High Priority</td></tr>
</table>

[45] In Level 1 Bridge Scour Inspection, Scour Condition Rating ScCR 3 is not assigned, yet the bridge is recommended to Proceed to Level 2 Scour Inspection

L.3.1 Comparison between method B1 and B2a

The comparison of the results indicate that all 25 bridges are ranked as PR 3 Move to Stage 2 Analysis according to Bekić-McKeough Method B1. This result is expected as Method B1, similar as UK BD 92/12 (BA74/06) is a staged process, and Method B2a is Stage 2 of Method B1. In other words, Method B1 recommends further analysis so that more precise and more reliable bridge rating can be assigned.

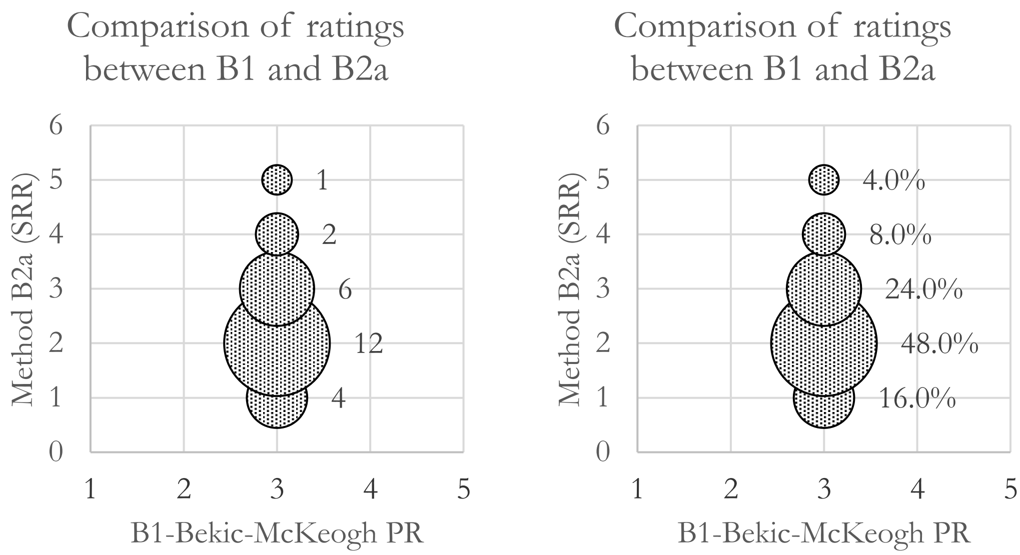


Figure 11.80 Results of the comparison between Method B1 and B2a for Data block 3.

L.3.2 Comparison between method L1 and B2a

Identically as for the results for B1 indicate that all 25 bridges are ranked “Proceed to Level 2” according to general bridge scour inspection from new inspection module. Methods B1 and L1 have strong correlation, and L1 Scour Condition rating 3 means that the bridge should be accessed with L2 inspection.

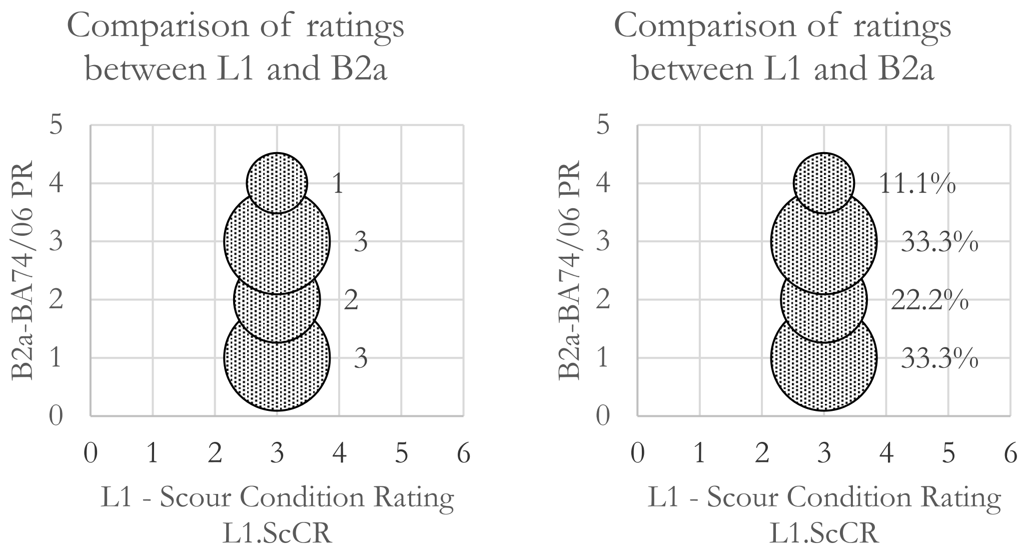
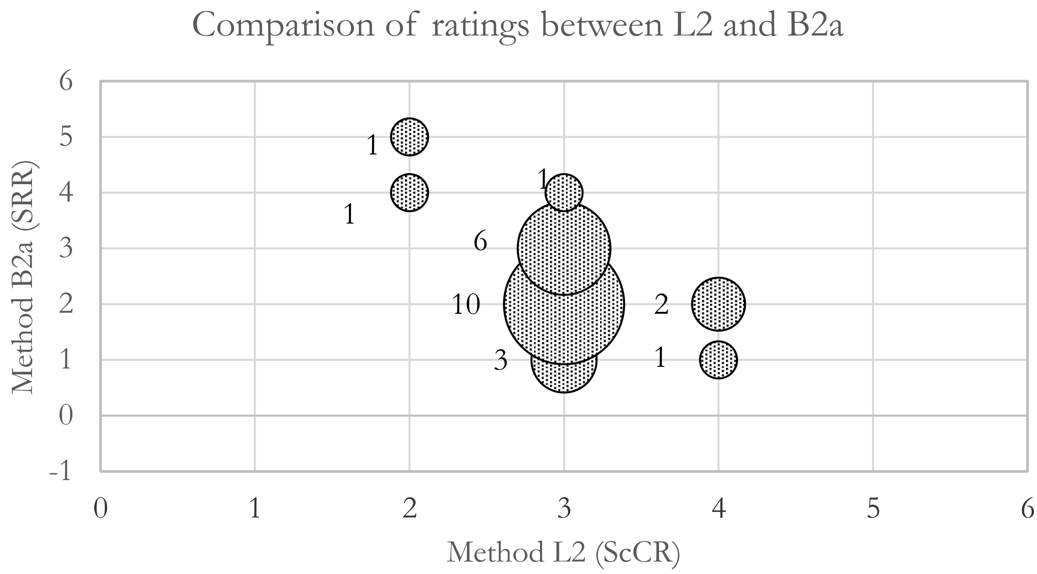


Figure 11.81 Results of the comparison between Method L1 and B2a for Data block 3.

L.3.3 Comparison between method L2 and B2a

The comparison of the results indicates significant differences between method L2 and methods B1 and L2. When comparing the results of the L2 scour condition rating L2.ScCR with B2a Priority Rating, the one can observe a weak correlation (R2 = 0.34). In the conclusions (see section 4.3) it was noted that the weakest point in method B2a is calculation of scour depth, which is often overestimated when compared to the observations [143-145]. Further, only 4 (16%) of 25 bridges have information on foundations. Due to fact that the estimated scour depth in Method B2a is unreliable and the depth of foundations are unknown for 84% of bridges it is considered that the comparison, e.g. Method B2a is unreliable. Method L2 provides an option for more accurate whilst conservative estimation of foundation depths in case foundations are unknown. Therefore Method L2 is superior to Method B2a.



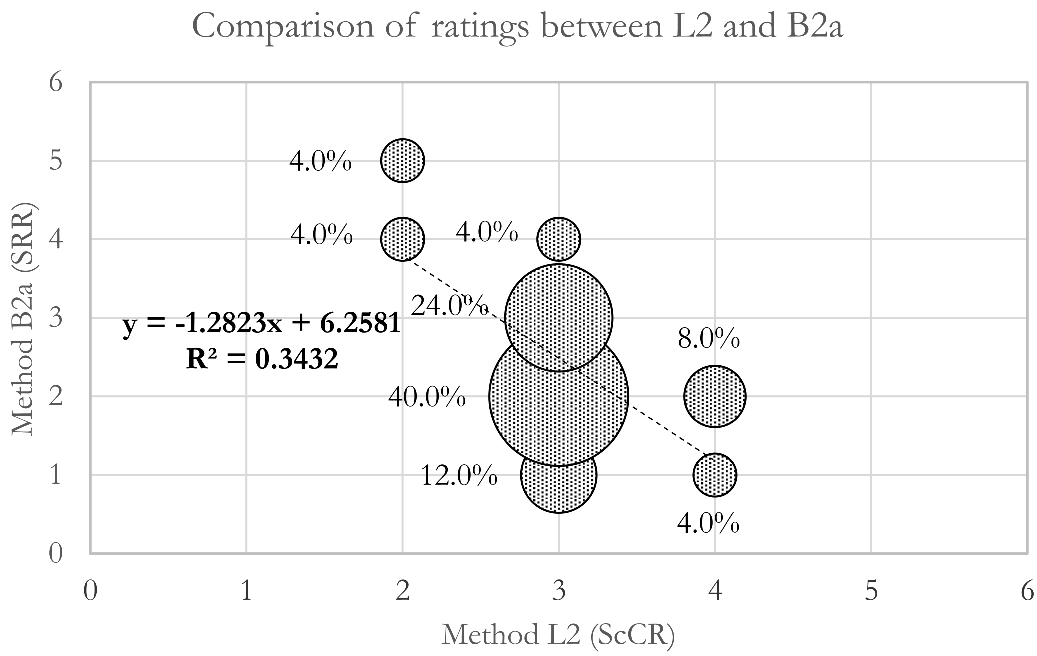
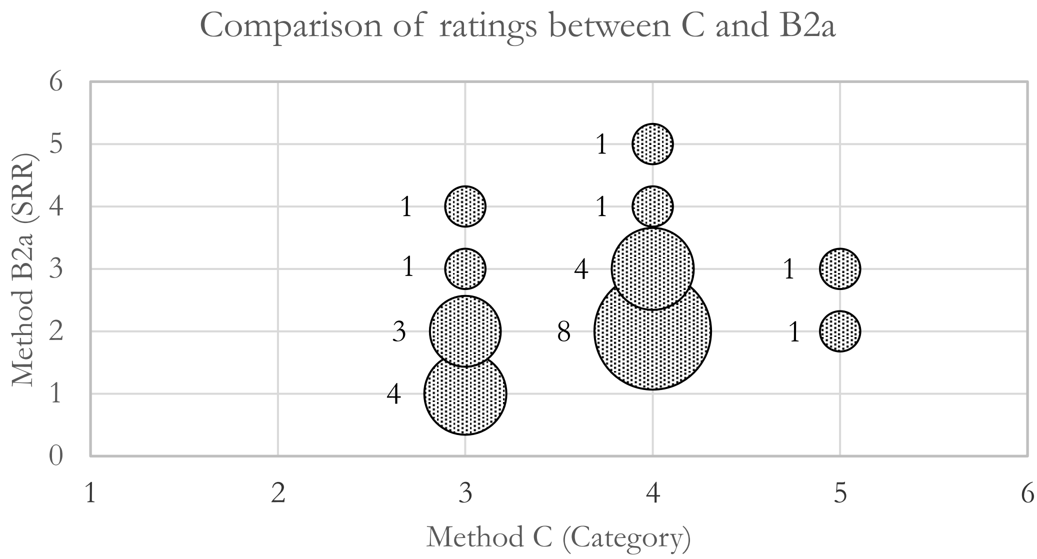


Figure 11.82 Results of the comparison between Method L2 and B2a for Data block 3.

L.3.4 Comparison between method C and B2a

There is no significant correlation between Methods B2a and C. Method C was marked as unreliable in the analysis conducted in section L.1 and L.2. The above section highlited that method B2a provides unreliable calculation of scour depth [143-145] and that only 4 (16%) of 25 bridges have information on foundations. Due to that results of Method B2a cannot be considered reliable.



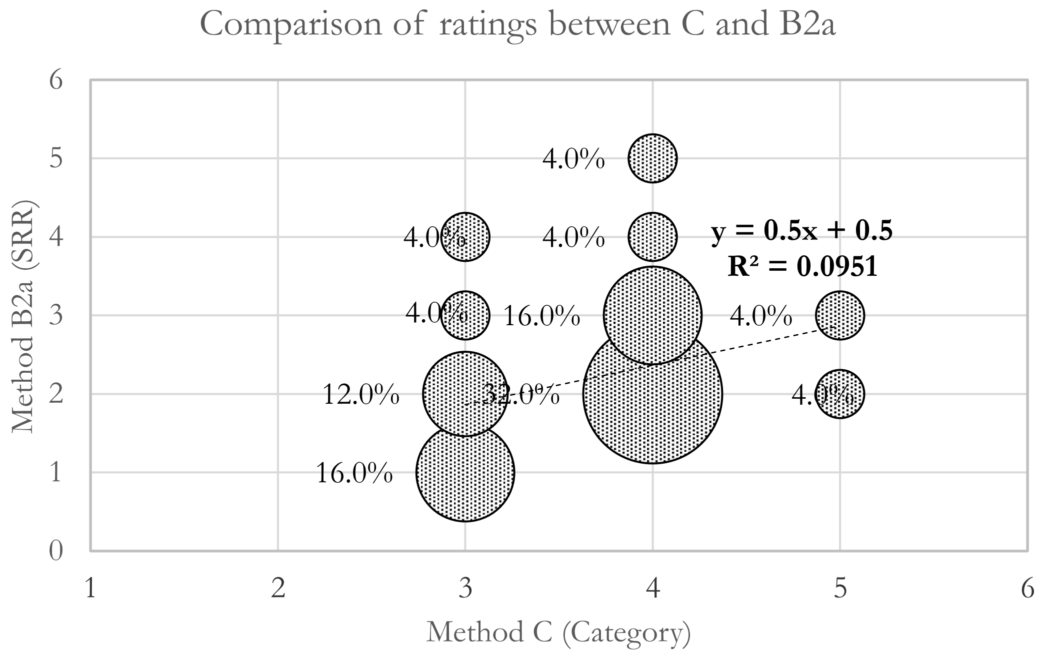


Figure 11.83 Results of the comparison between Method C and B2a for Data block 3.

L.3.5 Comparison between method B1 and B2b

The comparison of the results indicate that all 25 bridges are ranked as PR 3 (Move to Stage 2 Analysis) according to Bekić-McKeough Method B1. This is an expected result as Method B1, similar as UK BD 92/12 (BA74/06) is a staged process, and Method B2b is Stage 2 of Method B1.

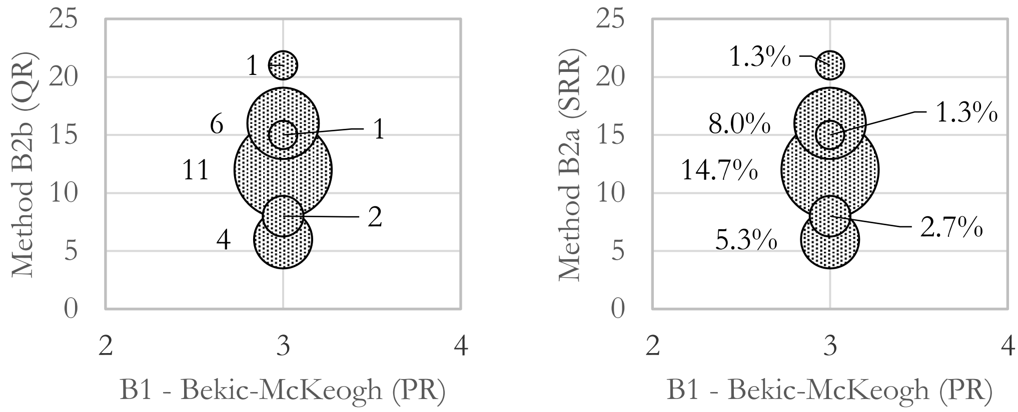


Figure 11.84 Results of the comparison between Method B1 and B2a for Data block 3.

L.3.6 Comparison between method L1 and B2b

Identically as for the results for B1 indicate that all 25 bridges are ranked "Proceed to Level 2" according to general bridge scour inspection from the new inspection module. Methods B1 and L1 have strong correlation, and L1 Scour Condition rating 3 means that the bridge should be accessed with L2 inspection.

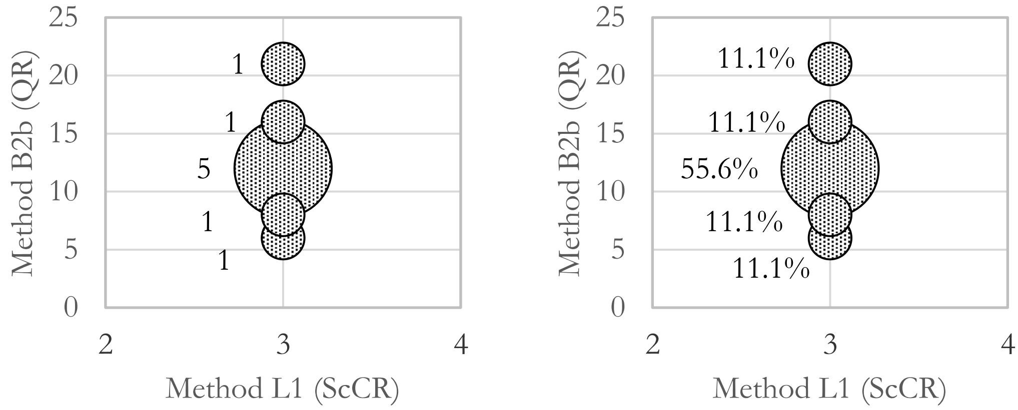


Figure 11.85 Results of the comparison between Method L1 and B2b for Data block 3.

L.3.7 Comparison between method L2 and B2b

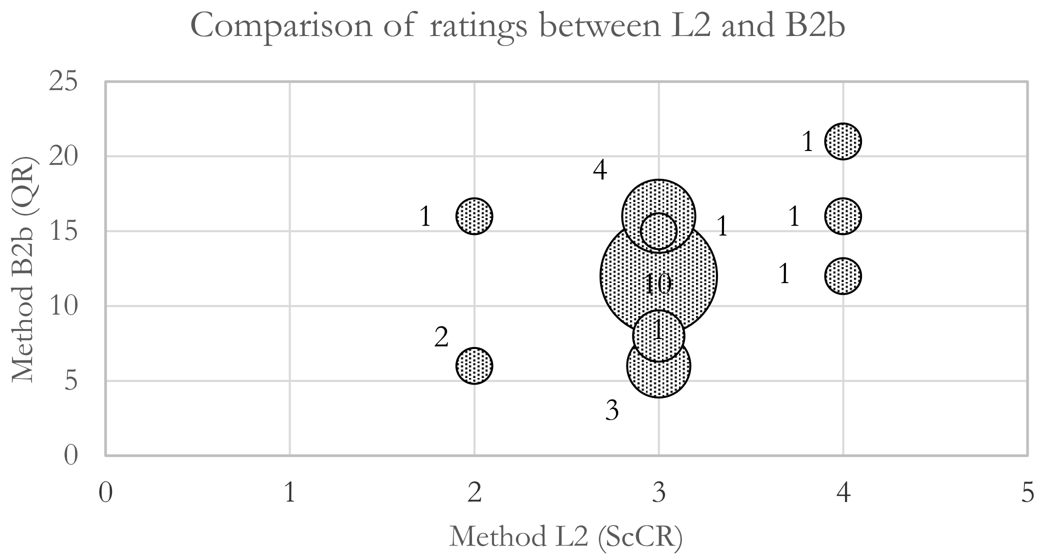
The correlation between the Method L2 and Method B2b inspection application results on 25 railway bridges in Ireland is very weak ($R^2 = 0.12$). For three bridges, Method L2 gives ScCR = 4 (damage is Critical), while Method B2b assigns QR of 12, 16 and 21 (undesirable) respectively. This result is acceptable according to Table 11.54. The largest dispersion between inspection results for 19 bridges rated with Method L2 ScCR = 3 (lower risk). Of this 19 bridges, 15 bridges have acceptable and correct result Table 11.54. However, four bridges of nineteen (21%) have unacceptable result using Method B2b, which assigns tolerable risk for those bridges. The most notable difference is for one bridge which is has ScCR = 2 from Method L2 (lower risk) and which is quantified as Qualitative rating of 16 (Intolerable risk) when using Method B2b. This result is unacceptable.

Comparison of results shows that methods have unacceptable differences for 5 bridges (20%), which is a better result than coefficient of determination R^2 might suggest.

One of the most significant differences between methods L2 and B2b is that Method B2b adds a costing variable into an equation, which might explain why some of the bridges are underrated when compared to method L2.

The costing component is important for bridge management, however from the point of view of safety to traffic over the bridge, if the bridge fails traffic should be stopped independently of the costing priority.

If needed, method L2 can be further improved by adding the costing component as one additional variable for safety of the bridge. For the analysis of safety of the bridge, the recommended method L2 is adequate.



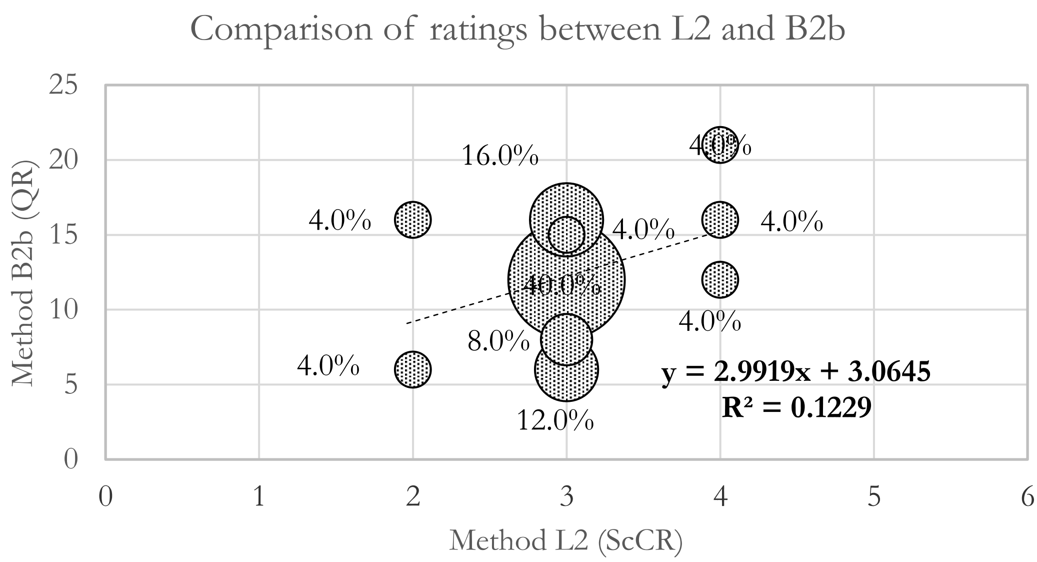


Figure 11.86 Results of the comparison between Method L2 and B2b for Data block 3.

L.3.8 Comparison between method C and B2b

Correlation between the Method L2 and Method C inspection application results on 25 railway bridges in Ireland is very weak (R^2 = 0.05). Comparison of results shows that methods have unacceptable differences for 12 bridges (48%), which is in accordance with low coefficient of determination R^2 = 0.05.

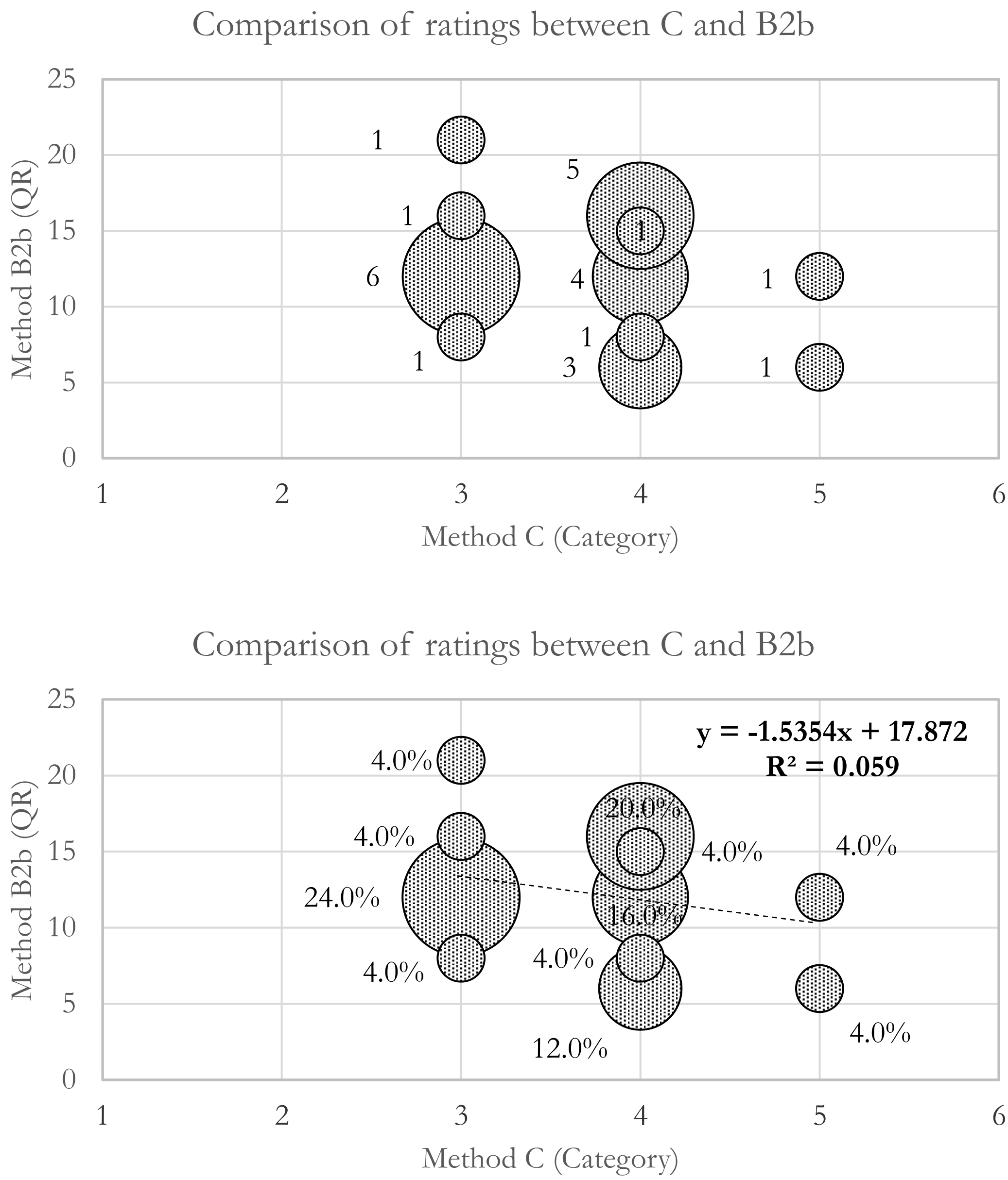
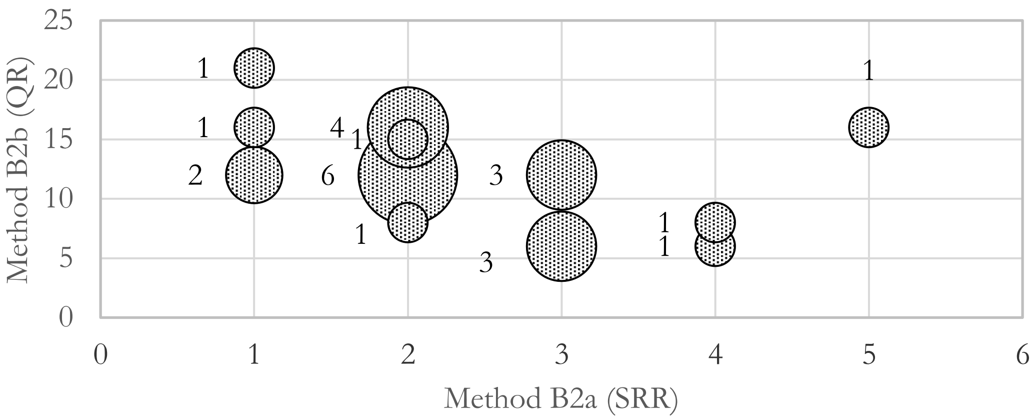


Figure 11.87 Results of the comparison between Method C and B2b for Data block 3.

L.3.9 Comparison between method B2a and B2b

The Priority Rating from Method B2a (UK BA 74/06 Stage 2) is compared with Method B2b Qualitative risk in Figure 11.88. The results indicate weak correlation ($R^2 = 0.19$) between Method B2a and Method B2b [162]. Priority rating and Qualitative Risk differ in principle. Priority Rating does not include the Severity of bridge Collapse and Qualitative Risk does not account for actual scour depth. Furthermore, equations for calculation of potential scour depths tend to overestimate the extent of scour. This raises a doubt if methods B2a and B2b are reliable methods for application. Comparison of results shows that the methods have unacceptable differences for 7 bridges (28%), which is in accordance with a low coefficient of determination $R^2 = 0.19$.



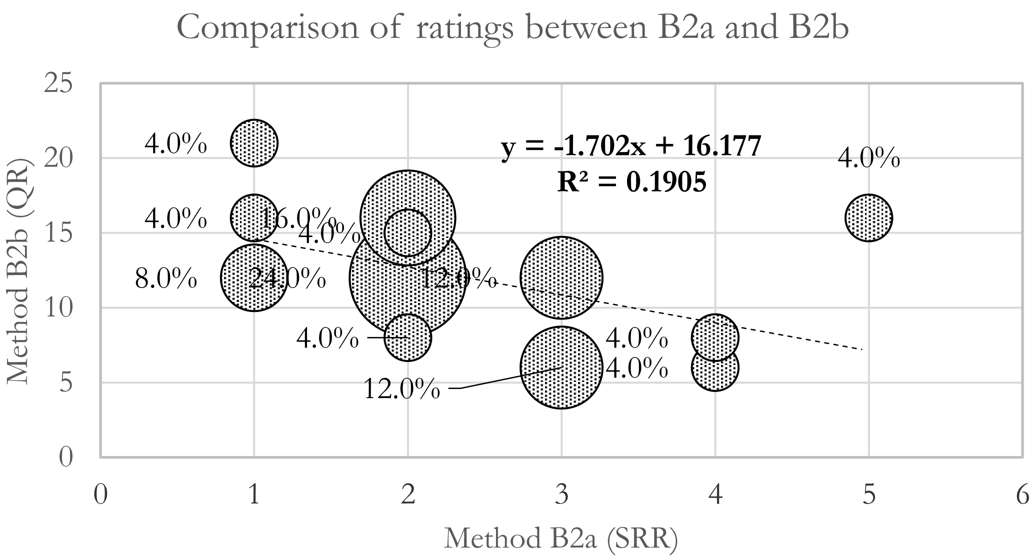


Figure 11.88 Results of the comparison between Method B2a and B2b for Data block 2.

L.4 Test for acceptable and unacceptable results

Additional tests of the number and percentage of acceptable and not acceptable outcomes of comparison between methods were conducted. The acceptable outcomes from the comparison are defined in tables (see Table 11.52, Table 11.53 and Table 11.54). In case that the percentage of non-acceptable outcomes is larger than 5%, methods will not be considered to be significantly correlated.

Table 11.55 counts number of acceptable and unacceptable results for the comparison between the methods for all three data blocks.

Table 11.56 shows the percentage of acceptable and unacceptable results for each of the comparisons. From this analysis it can be seen that comparable methods with better fit are Methods B1, L1 and L2. Method B2b seems to be somewhat comparable with method L1 as percentage of unacceptable results is 20%.

Comparison is unsatisfactory for the methods B2a, B2b and C. This test confirmed low correlation coefficients from the previous section.

Table 11.55. Number of acceptable and unacceptable results between comparison of methods

Data block 1 – 44 bridge

Comparison between methods		No. of acceptable results	No. of unacceptable results
B1	L1	44	0
	L2	44	0
	C*	28	15
L1	L2	44	0
	C*	18	25

Data block 2 – 101 bridge

Comparison between methods		No. of acceptable results	No. of unacceptable results
B1	L2	97	4
	C**	61	39
L2	C**	58	42

Data block 3 – 25 bridge

Comparison between methods		No. of acceptable results	No. of unacceptable results
B2a	B1	25	0
	L1***	9	0
	L2	14	11
	C	9	16
	B2b	17	8
B2b	B1	25	0
	L1***	9	0
	L2	20	5
	C	13	12
	B2a	17	8

*Sample of 43 bridges used; ** Sample of 100 bridges; *** Sample of 9 bridges

Table 11.56. Percentage of acceptable and unacceptable results between comparison of methods

Data block 1

Comparison between methods		Percentage acceptable results	Percentage unacceptable results	Mark and comment
B1	L1	100.0%	0.0%	Method L1 recommended
	L2	100.0%	0.0%	Method L2 recommended
	C*	65.1%	34.9%	Method C not recommended
L1	L2	100.0%	0.0%	Method L2 recommended
	C*	41.9%	58.1%	Method C not recommended

Data block 2

Comparison between methods		Percentage acceptable results	Percentage unacceptable results	Mark and comment
B1	L2	96.0%	4.0%	Method L2 recommended
	C**	61.0%	39.0%	Method C not recommended
L2	C**	58.0%	42.0%	Method C not recommended

Data block 3

Comparison between methods		Percentage acceptable results	Percentage of unacceptable results	Mark and comment
B2a	B1	100.0%	0.0%	Neutral - comparison cannot be made
	L1***	100.0%	0.0%	Neutral - comparison cannot be made
	L2	56.0%	44.0%	Results NOT satisfactory
	C	36.0%	64.0%	Results NOT satisfactory
	B2b	68.0%	32.0%	Results NOT satisfactory
B2b	B1	100.0%	0.0%	Neutral - comparison cannot be made
	L1***	100.0%	0.0%	Neutral - comparison cannot be made
	L2	80.0%	20.0%	Results between L2 and B2b satisfactory
	C	52.0%	48.0%	Results NOT satisfactory
	B2a	68.0%	32.0%	Results NOT satisfactory

*Sample of 43 bridges used; ** Sample of 100 bridges; *** Sample of 9 bridges

Annex M Correlation analysis

M.1 Input data

M.1.1 Data Block 1

	Bridge Name	B1 PR	L1 ScCR	L2 ScCR	C Category
1	UB110a	1	0	0	5
2	UB116a	1	0	0	5
3	UB125-Lim/Tuam	2	1	2	5
4	UB126	2	1	2	4
5	UB130	2	2	2	3
6	UB145	1	0	0	5
7	UB15	2	2	2	4
8	UB150	2	2	2	5
9	UB154	3	3	3	3
10	UB155	3	3	3	4
11	UB190	3	3	3	3
12	UB191	2	2	2	4
13	UB193	2	2	2	3
14	UB199	3	3	3	3
15	UB207	3	3	4	3
16	UB208	2	2	2	4
17	UB221	2	2	2	4
18	UB244	2	2	2	4
19	UB25	2	2	2	4
20	UB295	2	2	2	4
21	UB30	3	3	3	3
22	UB300	2	1	2	4
23	UB303	2	2	2	3
24	UB309	3	3	3	4
25	UB314	2	2	2	4
26	UB320	2	2	2	3
27	UB101a	1	0	0	4
28	UB321	2	2	2	4
29	UB342	3	3	3	5
30	UB35	2	2	1	5
31	UB39	2	2	2	3
32	UB39	2	2	2	5
33	UB53	1	0	0	4

34	UB539	2	2	1	3
35	UB57	2	2	2	4
36	UB594	2	1	2	5
37	UB63	2	2	2	4
38	UB65-Lim/Enn	2	2	2	5
39	UB743	2	1	1	4
40	UB766	2	1	1	4
41	UB79	2	1	2	4
42	UB79 Lim/Wat	2	1	2	4
43	UB796	2	2	1	4
44	UB97	3	3	3	4

M.1.2 Data block 2

	Bridge Name	*B1 PR*	*L1 ScCR*	*L2 ScCR*	*C Category*
1	UB110a	1	0	0	5
2	UB116a	1	0	0	5
3	UB125-Lim/Tuam	2	1	2	5
4	UB126	2	1	2	4
5	UB130	2	2	2	3
6	UB145	1	0	0	5
7	UB15	2	2	2	4
8	UB150	2	2	2	5
9	UB154	3	3	3	3
10	UB155	3	3	3	4
11	UB190	3	3	3	3
12	UB191	2	2	2	4
13	UB193	2	2	2	3
14	UB199	3	3	3	3
15	UB207	3	3	4	3
16	UB208	2	2	2	4
17	UB221	2	2	2	4
18	UB244	2	2	2	4
19	UB25	2	2	2	4
20	UB295	2	2	2	4
21	UB30	3	3	3	3
22	UB300	2	1	2	4
23	UB303	2	2	2	3
24	UB309	3	3	3	4
25	UB314	2	2	2	4
26	UB320	2	2	2	3
27	UB101a	1	0	0	4
28	UB321	2	2	2	4
29	UB342	3	3	3	5

30	UB35	2	2	1	5
31	UB39	2	2	2	3
32	UB39	2	2	2	5
33	UB53	1	0	0	4
34	UB539	2	2	1	3
35	UB57	2	2	2	4
36	UB594	2	1	2	5
37	UB63	2	2	2	4
38	UB65-Lim/Enn	2	2	2	5
39	UB743	2	1	1	4
40	UB766	2	1	1	4
41	UB79	2	1	2	4
42	UB79 Lim/Wat	2	1	2	4
43	UB796	2	2	1	4
44	UB97	3	3	3	4
45	UB01	3		2	4
46	UB104	2		2	4
47	UB106	2		1	4
48	UB111	3		3	4
49	UB125-Dub/Gal	2		2	4
50	UB134	2		2	4
51	UB139	3		3	4
52	UB140	3		2	4
53	UB147	2		2	4
54	UB149	2		2	4
55	UB159	2		2	4
56	UB16	2		2	4
57	UB160	3		4	4
58	UB168	3		3	4
59	UB170	2		2	4
60	UB178	3		4	3
61	UB18	3		3	4
62	UB18	2		1	4
63	UB180	3		3	4
64	UB181	3		3	3
65	UB186	3		3	3
66	UB188	3		3	3
67	UB19	3		3	5
68	UB198	3		3	3
69	UB1B	1		0	4
70	UB247	2		2	4
71	UB262	2		2	4
72	UB28	3		3	4
73	UB296 Dub/Crk	2		2	3
74	UB296 Dub/Wex	2		2	4

75	UB3	2		2	4
76	UB31	2		2	4
77	UB32	2		2	4
78	UB323	3		3	4
79	UB341	3		3	4
80	UB36	4		5	4
81	UB391	2		2	4
82	UB40	2		2	4
83	UB413	2		2	3
84	UB44	1		0	6
85	UB45	2		3	4
86	UB47	3		3	4
87	UB500	2		3	4
88	UB56	1		0	6
89	UB588	2		2	4
90	UB599	2		1	4
91	UB65 Mllw/Tra	2		2	4
92	UB65-Dub/Bel	3		3	4
93	UB7	1		0	4
94	UB72	3		3	4
95	UB72	2		2	6
96	UB728	3		3	4
97	UB77	3		3	4
98	UB82	1		0	6
99	UB887	2		2	4
100	UB93	3		3	4
101	UB95	4		4	4

M.1.3 Data block 3

	Bridge Name	B1 PR	L1 ScCR	L2 ScCR	C Category	B2a PR	B2b QR
1	UBC309	3	3	3	4	2	16
2	UBE154	3	3	3	3	2	12
3	UBE30	3	3	3	3	4	8
4	UBE97	3	3	3	4	3	12
5	UBG155	3	3	3	4	3	12
6	UBR190	3	3	3	3	1	12
7	UBR199	3	3	3	3	1	12
8	UBR207	3	3	4	3	1	21
9	UBR342	3	3	3	5	3	6
10	UB01	3	-	2	4	4	6
11	UBB19	3	-	3	5	2	12
12	UBC168	3	-	3	4	2	16
13	UBE77	3	-	3	4	3	6
14	UBH140	3	-	2	4	5	16
15	UBH180	3	-	3	4	2	15
16	UBK47	3	-	3	4	2	16
17	UBM728	3	-	3	4	2	12
18	UBN93	3	-	3	4	2	8
19	UBR160	3	-	4	4	2	16
20	UBR178	3	-	4	3	2	12
21	UBR181	3	-	3	3	2	12
22	UBR186	3	-	3	3	3	12
23	UBR188	3	-	3	3	1	16
24	UBR323	3	-	3	4	2	12
25	UBT28	3	-	3	4	3	6

M.2 Correlation analysis

M.2.1 Data block 1

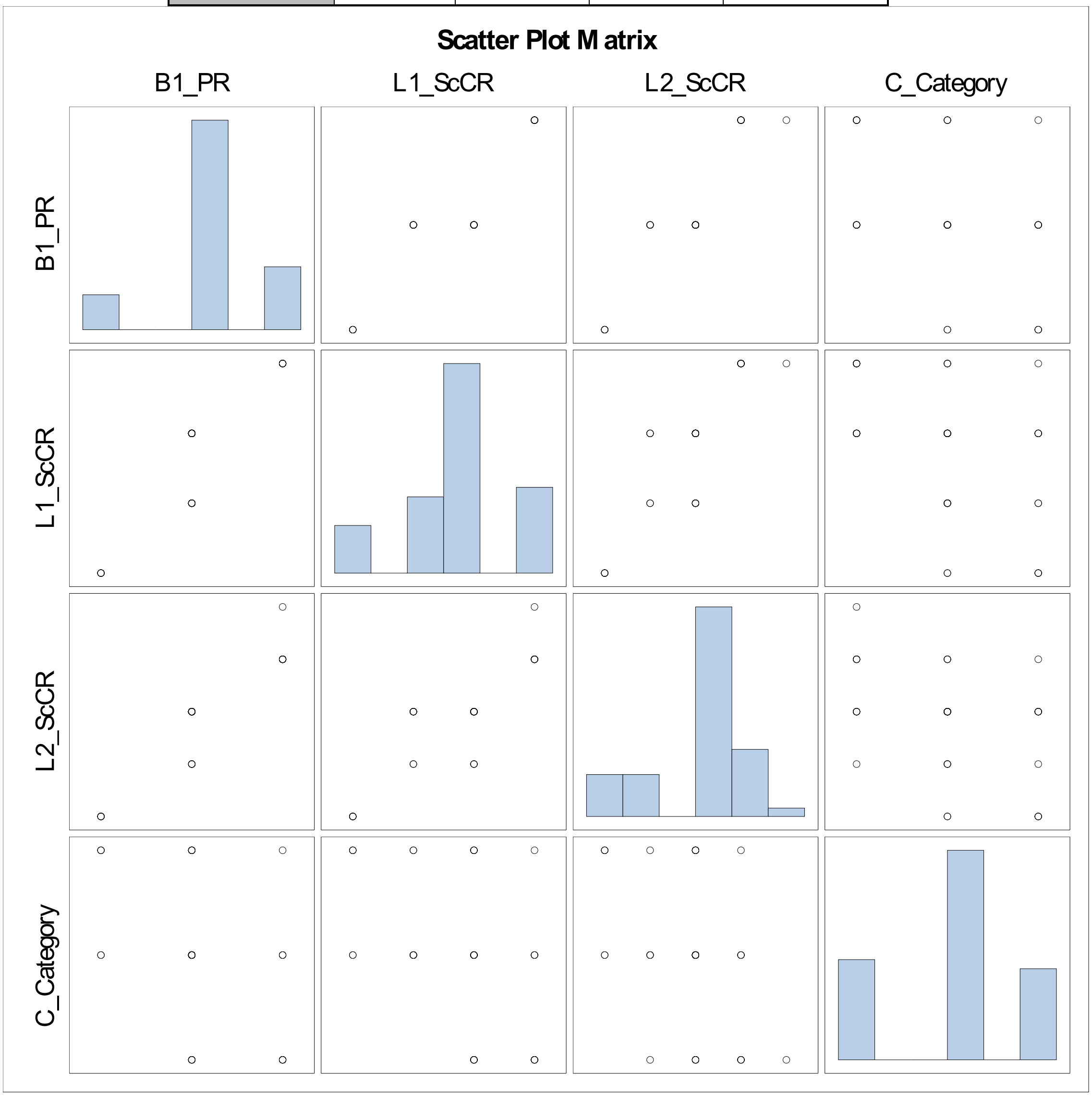
Simple Statistics							
Variable	**N**	**Mean**	**Std Dev**	**Median**	**Minimum**	**Maximum**	**Label**
B1_PR	44	2.09091	0.56314	2.00000	1.00000	3.00000	B1 PR
L1_ScCR	44	1.79545	0.90424	2.00000	0	3.00000	L1 ScCR
L2_ScCR	44	1.88636	0.92046	2.00000	0	4.00000	L2 ScCR
C_Category	44	3.97727	0.69846	4.00000	3.00000	5.00000	C Category

Pearson Correlation Coefficients, N = 44 Prob > |r| under H0: Rho=0				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	1.00000	0.90510 <.0001	0.91770 <.0001	-0.40850 0.0059
L1_ScCR	0.90510 <.0001	1.00000	0.86553 <.0001	-0.44940 0.0022
L2_ScCR	0.91770 <.0001	0.86553 <.0001	1.00000	-0.40201 0.0068
C_Category	- 0.40850 0.0059	-0.44940 0.0022	-0.40201 0.0068	1.00000

Spearman Correlation Coefficients, N = 44 Prob > |r| under H0: Rho=0				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	1.00000	0.88517 <.0001	0.91287 <.0001	-0.40679 0.0061
L1_ScCR	0.88517 <.0001	1.00000	0.83341 <.0001	-0.44468 0.0025
L2_ScCR	0.91287 <.0001	0.83341 <.0001	1.00000	-0.37862 0.0113
C_Category	-0.40679 0.0061	-0.44468 0.0025	-0.37862 0.0113	1.00000

Kendall Tau b Correlation Coefficients, N = 44 Prob > |tau| under H0: Tau=0				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	1.00000	0.85172 <.0001	0.88181 <.0001	-0.38087 0.0065
L1_ScCR	0.85172 <.0001	1.00000	0.78336 <.0001	-0.40225 0.0028
L2_ScCR	0.88181 <.0001	0.78336 <.0001	1.00000	-0.34257 0.0114
C_Category	-0.38087 0.0065	-0.40225 0.0028	-0.34257 0.0114	1.00000

Hoeffding Dependence Coefficients, N = 44 Prob > D under H0: D=0				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	0.10972 <.0001	0.11405 <.0001	0.11411 <.0001	-0.00074 0.3933
L1_ScCR	0.11405 <.0001	0.30699 <.0001	0.13673 <.0001	0.01824 0.0653
L2_ScCR	0.11411 <.0001	0.13673 <.0001	0.24681 <.0001	0.00177 0.3035
C_Category	-0.00074 0.3933	0.01824 0.0653	0.00177 0.3035	0.24090 <.0001



M.2.2 Data block 2

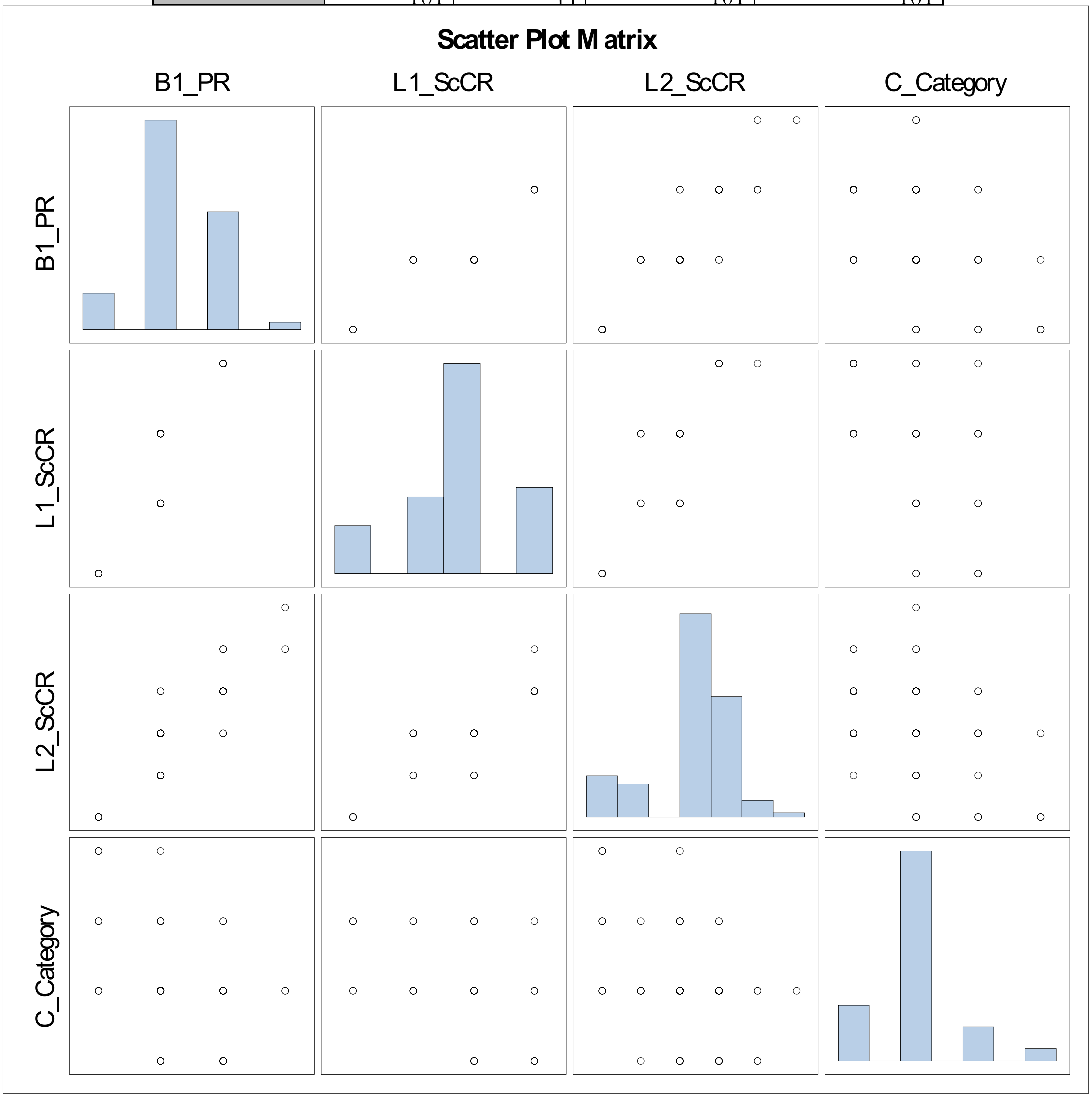
Simple Statistics							
Variable	**N**	**Mean**	**Std Dev**	**Median**	**Minimum**	**Maximum**	**Label**
B1_PR	101	2.25743	0.65808	2.00000	1.00000	4.00000	B1 PR
L1_ScCR	44	1.79545	0.90424	2.00000	0	3.00000	L1 ScCR
L2_ScCR	101	2.11881	1.00287	2.00000	0	5.00000	L2 ScCR
C_Category	101	4.00990	0.67075	4.00000	3.00000	6.00000	C Category

Pearson Correlation Coefficients Prob > |r| under H0: Rho=0 Number of Observations				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	1.00000 101	0.90510 <.0001 44	0.90779 <.0001 101	-0.39097 <.0001 101
L1_ScCR	0.90510 <.0001 44	1.00000 44	0.86553 <.0001 44	-0.44940 0.0022 44
L2_ScCR	0.90779 <.0001 101	0.86553 <.0001 44	1.00000 101	-0.41802 <.0001 101
C_Category	-0.39097 <.0001 101	-0.44940 0.0022 44	-0.41802 <.0001 101	1.00000 101

Spearman Correlation Coefficients Prob > |r| under H0: Rho=0 Number of Observations				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	1.00000 101	0.88517 <.0001 44	0.90710 <.0001 101	-0.35600 0.0003 101
L1_ScCR	0.88517 <.0001 44	1.00000 44	0.83341 <.0001 44	-0.44468 0.0025 44
L2_ScCR	0.90710 <.0001 101	0.83341 <.0001 44	1.00000 101	-0.35617 0.0003 101
C_Category	-0.35600 0.0003 101	-0.44468 0.0025 44	-0.35617 0.0003 101	1.00000 101

Kendall Tau b Correlation Coefficients Prob > |tau| under H0: Tau=0 Number of Observations				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	1.00000 101	0.85172 <.0001 44	0.87634 <.0001 101	-0.33086 0.0003 101
L1_ScCR	0.85172 <.0001 44	1.00000 44	0.78336 <.0001 44	-0.40225 0.0028 44
L2_ScCR	0.87634 <.0001 101	0.78336 <.0001 44	1.00000 101	-0.32164 0.0003 101
C_Category	-0.33086 0.0003 101	-0.40225 0.0028 44	-0.32164 0.0003 101	1.00000 101

Hoeffding Dependence Coefficients Prob > D under H0: D=0 Number of Observations				
	B1_PR	**L1_ScCR**	**L2_ScCR**	**C_Category**
B1_PR	0.18837 <.0001 101	0.11405 <.0001 44	0.17110 <.0001 101	0.00731 0.0726 101
L1_ScCR	0.11405 <.0001 44	0.30699 <.0001 44	0.13673 <.0001 44	0.01824 0.0653 44
L2_ScCR	0.17110 <.0001 101	0.13673 <.0001 44	0.30054 <.0001 101	0.00913 0.0506 101
C_Category	0.00731 0.0726 101	0.01824 0.0653 44	0.00913 0.0506 101	0.14231 <.0001 101



M.2.3 Data block 3

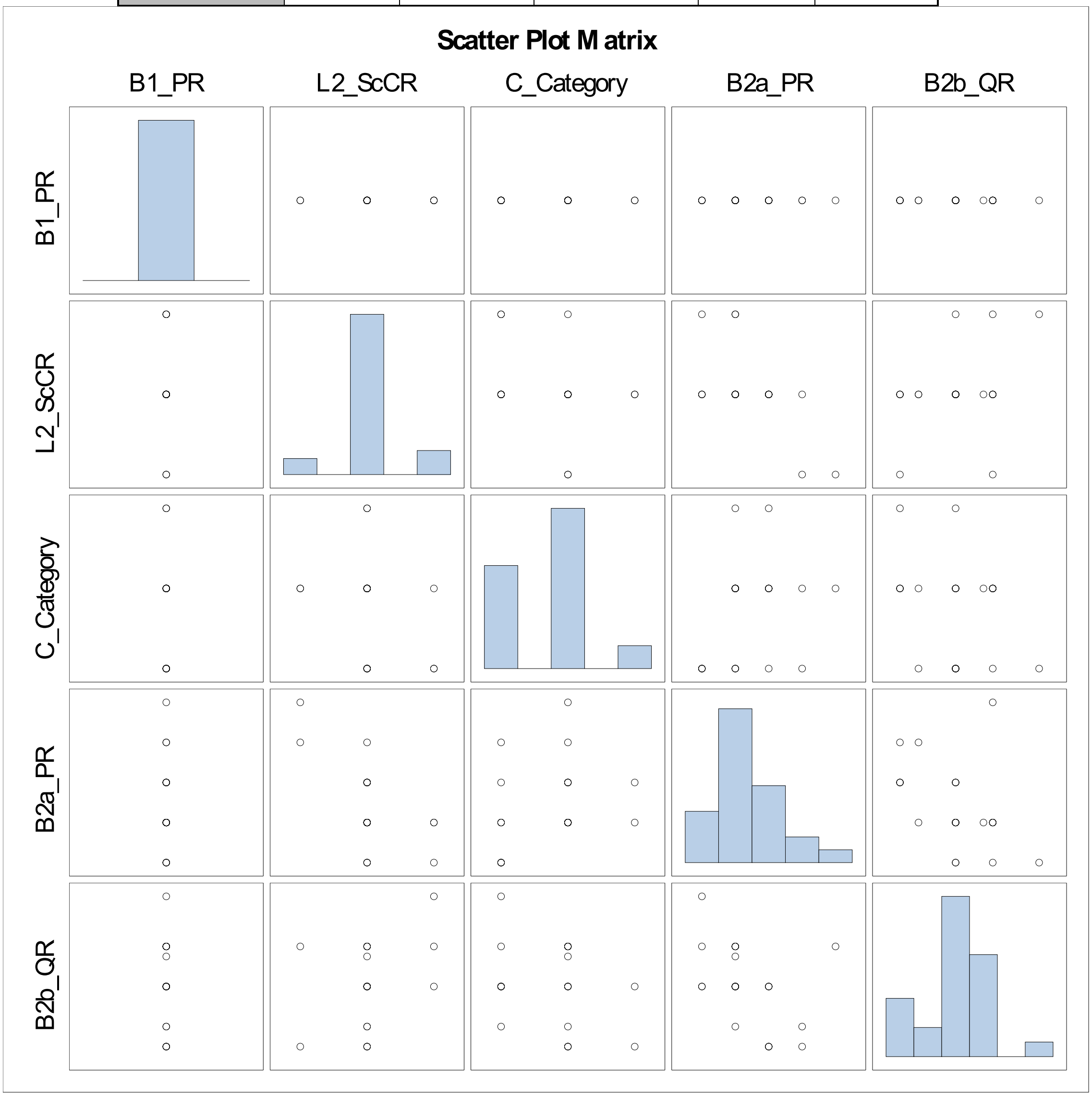
Simple Statistics							
Variable	**N**	**Mean**	**Std Dev**	**Median**	**Minimum**	**Maximum**	**Label**
B1_PR	25	3.00000	0	3.00000	3.00000	3.00000	B1 PR
L2_ScCR	25	3.04000	0.45461	3.00000	2.00000	4.00000	L2 ScCR
C_Category	25	3.72000	0.61373	4.00000	3.00000	5.00000	C Category
B2a_PR	25	2.36000	0.99499	2.00000	1.00000	5.00000	B2a PR
B2b_QR	25	12.16000	3.88029	12.00000	6.00000	21.00000	B2b QR

Pearson Correlation Coefficients, N = 25 Prob > |r| under H0: Rho=0					
	B1_PR	**L2_ScCR**	**C_Category**	**B2a_PR**	**B2b_QR**
B1_PR	1.00000 .	0	0.	0.	0.
L2_ScCR	0	1.00000	-0.25686 0.2152	-0.58586 0.0021	0.35053 0.0858
C_Category	0	-0.25686 0.2152	1.00000	0.30841 0.1336	-0.24285 0.2421
B2a_PR	0	-0.58586 0.0021	0.30841 0.1336	1.00000	-0.43643 0.0292
B2b_QR	0	0.35053 0.0858	-0.24285 0.2421	-0.43643 0.0292	1.00000

Spearman Correlation Coefficients, N = 25 Prob > |r| under H0: Rho=0					
	B1_PR	**L2_ScCR**	**C_Category**	**B2a_PR**	**B2b_QR**
B1_PR **B1 PR**	1.00000 .	0	0.	0.	0.
L2_ScCR **L2 ScCR**	0	1.00000	-0.27592 0.1819	-0.50271 0.0104	0.28662 0.1648
C_Category **C Category**	0	-0.27592 0.1819	1.00000	0.37876 0.0619	-0.15873 0.4485
B2a_PR **B2a PR**	0	-0.50271 0.0104	0.37876 0.0619	1.00000	-0.49484 0.0119
B2b_QR **B2b QR**	0	0.28662 0.1648	-0.15873 0.4485	-0.49484 0.0119	1.00000

Kendall Tau b Correlation Coefficients, N = 25 Prob > |tau| under H0: Tau=0					
	B1_PR	**L2_ScCR**	**C_Category**	**B2a_PR**	**B2b_QR**
B1_PR	1.00000 .	0	0.	0.	0.
L2_ScCR	0	1.00000	-0.25921 0.1805	-0.46696 0.0114	0.26017 0.1542
C Category	0	-0.25921 0.1805	1.00000	0.34039 0.0633	-0.13786 0.4471
B2a_PR	0	-0.46696 0.0114	0.34039 0.0633	1.00000	-0.43692 0.0116
B2b_QR	0	0.26017 0.1542	-0.13786 0.4471	-0.43692 0.0116	1.00000

Hoeffding Dependence Coefficients, N = 25 Prob > D under H0: D=0					
	B1_PR	**L2_ScCR**	**C_Category**	**B2a_PR**	**B2b_QR**
B1_PR	-0.09317 1.0000	-0.07680 1.0000	-0.06698 1.0000	-0.06382 1.0000	-0.06297 1.0000
L2_ScCR	-0.07680 1.0000	-0.00392 0.4576	-0.04631 1.0000	-0.02167 0.9865	-0.04550 1.0000
C_Category	-0.06698 1.0000	-0.04631 1.0000	0.12956 0.0009	-0.01533 0.8356	-0.03711 1.0000
B2a_PR	-0.06382 1.0000	-0.02167 0.9865	-0.01533 0.8356	0.31266 <.0001	0.02498 0.0951
B2b_QR	-0.06297 1.0000	-0.04550 1.0000	-0.03711 1.0000	0.02498 0.0951	0.39359 <.0001



M.3 Summary for the Correlation Analysis

All correlation coefficients used (see section 7.2) show very similar pattern. Parametric correlation analysis is shown in Table 11.57-Table 11.58 and non-parametric correlation analysis is shown in Table 11.59-Table 11.61. Strong correlation is confirmed between methods B1, L1 and L2. Correlation combinations between other methods show weak fit. From all remaining correlations, that are generally poor, there is some more moderate correlation between Method B2a and L2. The low "p" values for the correlation indicate that there is stronger correlation for all parameters.

Table 11.61 showing Hoeffding Dependance Coefficient generally show weak correlation. This is additionally confirmed with higher "p" values (see section M.2) when Method C was compared to Method L2, B2a and B2b, indicating that Method C should be dismissed.

Table 11.57. Results of evaluation showing Coefficient of determination R^2.

Method	B1	L1	L2	C	B2a	B2b
B1	1.0	0.82	0.82	0.17	0.00	0.00
L1	0.82	1.0	0.75	0.20	0.00	0.00
L2	0.82	0.75	1.0	0.17	0.34	0.12
C	0.17	0.20	0.17	1.0	0.10	0.06
B2a	0.00	0.00	0.34	0.10	1.0	0.19
B2b	0.00	0.00	0.12	0.06	0.19	1.0

Table 11.58. Results of evaluation showing Pearsons correlation coefficient "r".

Method	B1	L1	L2	C	B2a	B2b
B1		+0.91	+0.92	-0.41	0.00	0.00
L1	+0.91		+0.87	-0.45	0.00	0.00
L2	+0.92	+0.87		-0.42	-0.59	+0.35
C	-0.41	-0.45	-0.42		+0.31	-0.24
B2a	0.00	0.00	-0.59	+0.31		-0.44
B2b	0.00	0.00	+0.35	-0.24	-0.44	

Table 11.59. Results of evaluation showing Spearman's correlation coefficient.

Method	B1	L1	L2	C	B2a	B2b
B1		+0.89	+0.91	-0.36	0.00	0.00
L1	+0.89		+0.83	-0.44	0.00	0.00
L2	+0.91	+0.83		-0.36	-0.50	+0.29
C	-0.36	-0.44	-0.36		+0.38	-0.16
B2a	0.00	0.00	-0.50	+0.38		-0.49
B2b	0.00	0.00	+0.29	-0.16	-0.49	

Table 11.60. Results of evaluation showing Kendall's Tau-b correlation coefficient.

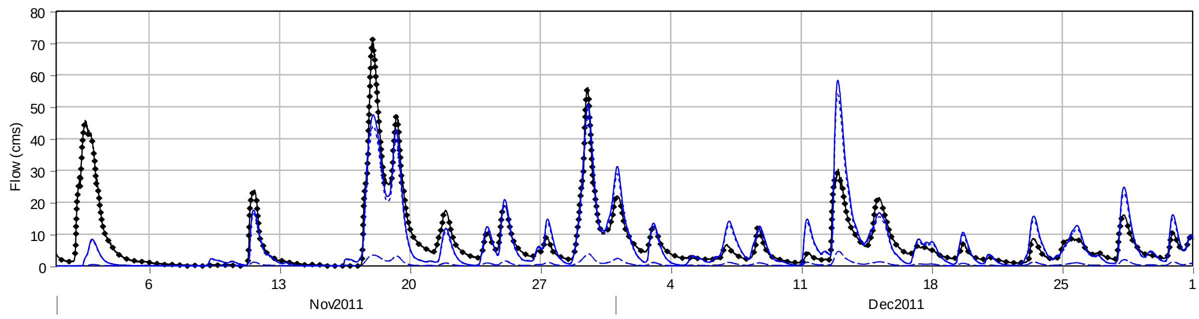
Method	B1	L1	L2	C	B2a	B2b
B1		+0.85	+0.88	-0.33	0.00	0.00
L1	+0.85		+0.78	-0.40	0.00	0.00
L2	+0.88	+0.78		-0.32	-0.47	+0.26
C	-0.33	-0.40	-0.32		+0.34	-0.14
B2a	0.00	0.00	-0.47	+0.34		-0.44
B2b	0.00	0.00	+0.26	-0.14	-0.44	

Table 11.61. Results of evaluation showing Hoeffding Dependence Coefficient.

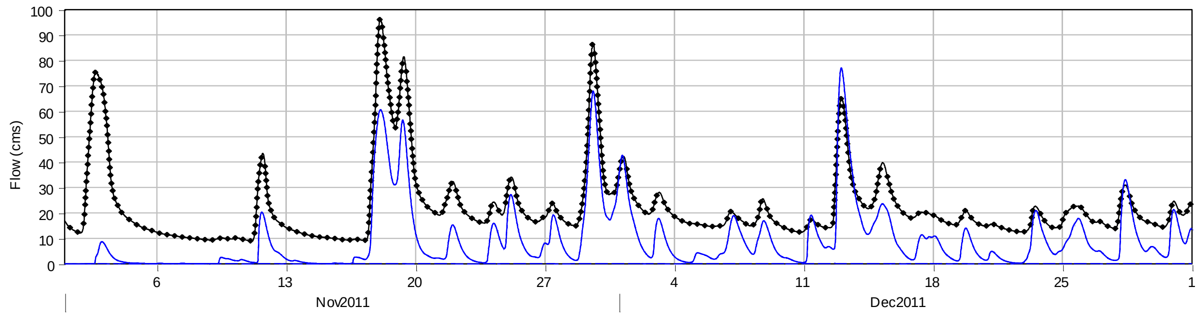
Method	B1	L1	L2	C	B2a	B2b
B1	0.19	0.11	0.17	0.01	-0.06	-0.06
L1	0.11	0.31	0.14	0.02	n/a	n/a
L2	0.17	0.14	0.30	0.01	-0.02	-0.05
C	0.01	0.02	0.01	0.14	-0.02	-0.04
B2a	-0.06	n/a	-0.02	-0.02	0.31	0.02
B2b	-0.06	n/a	-0.05	-0.04	0.02	0.39

Annex N Calibration results of Bandon HEC-HMS model

a) Long Bridge – HS20008



b) Bealaboy – HS20016



c) Bandon – HS20001

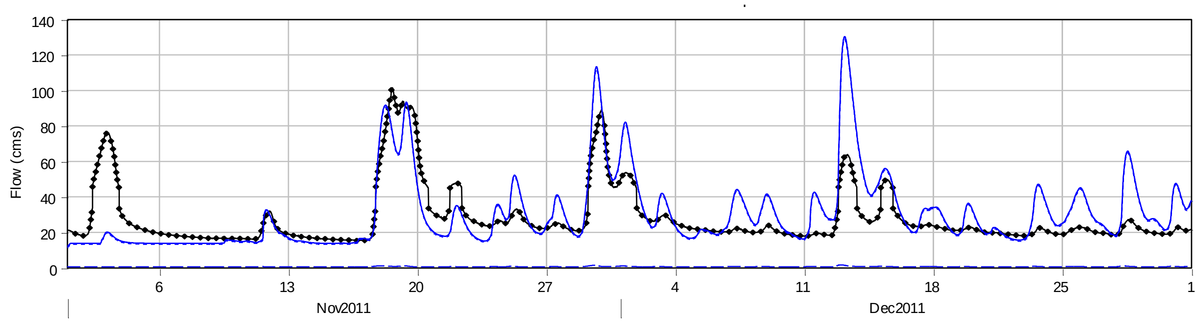
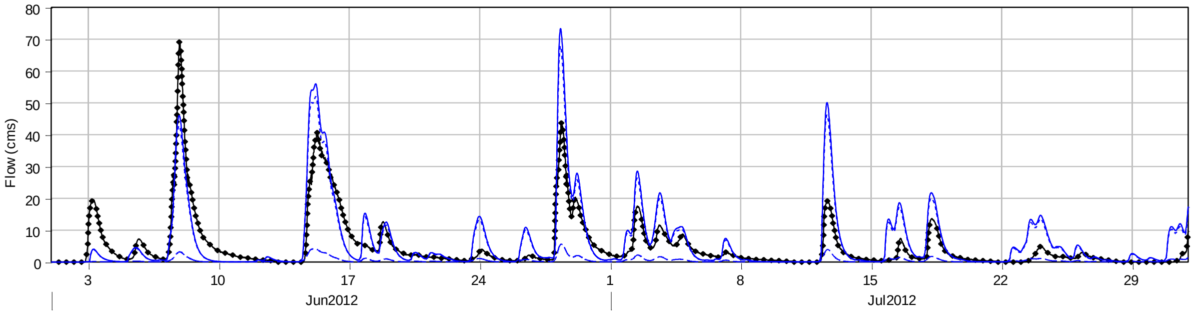
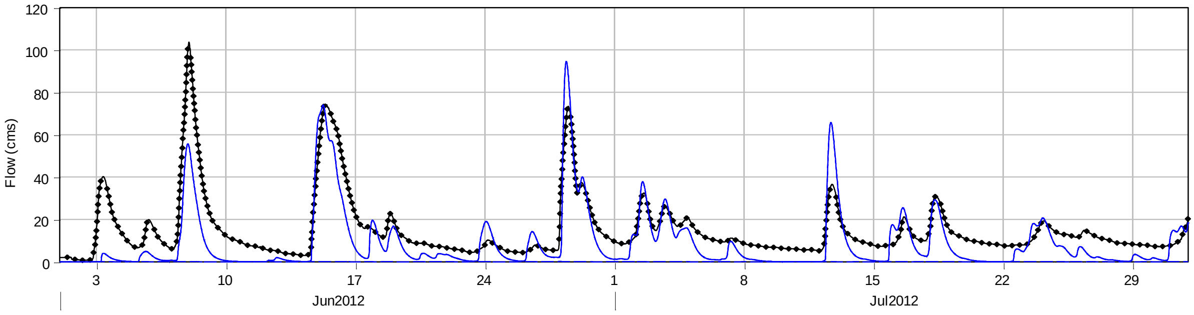


Figure 11.89. Calibration results for December 2011 (blue line - simulated; black line - observed).

a) Long Bridge – HS20008



b) Bealaboy – HS20016



c) Bandon – HS20001

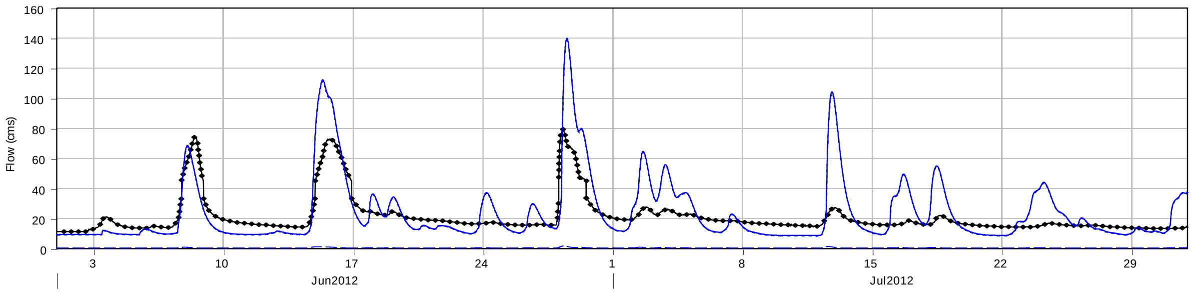
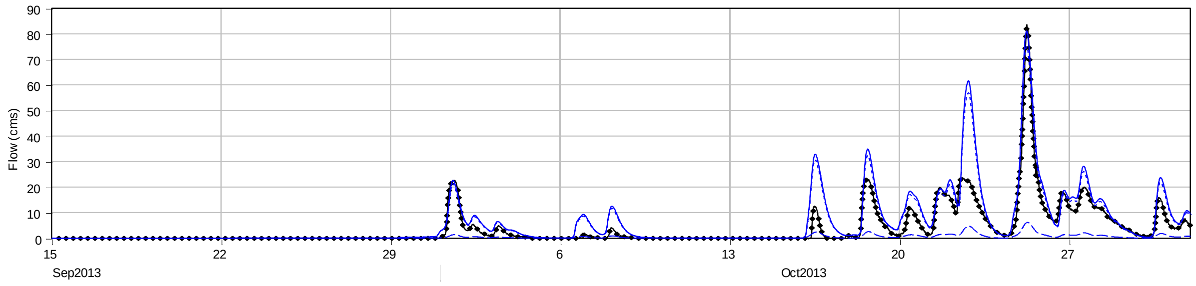
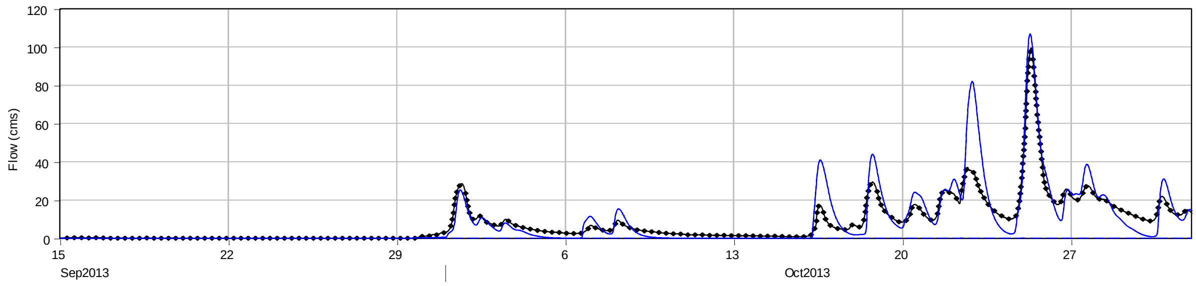


Figure 11.90. Calibration results for July 2012 (blue line - simulated; black line - observed).

a) Long Bridge – HS20008



b) Bealaboy – HS20016



c) Bandon – HS20001

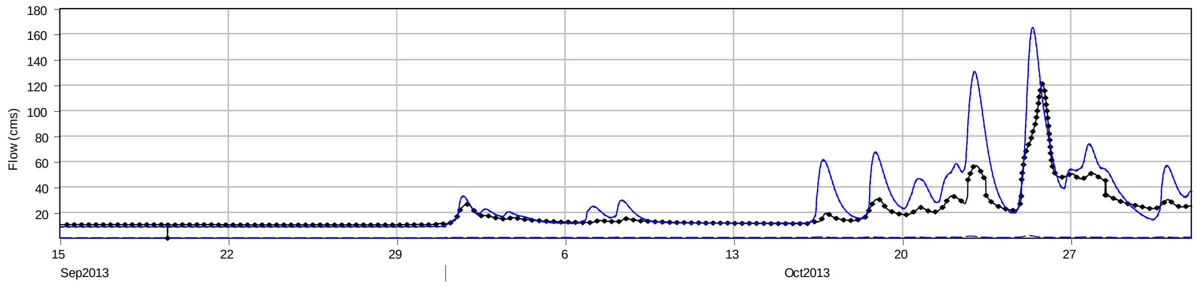
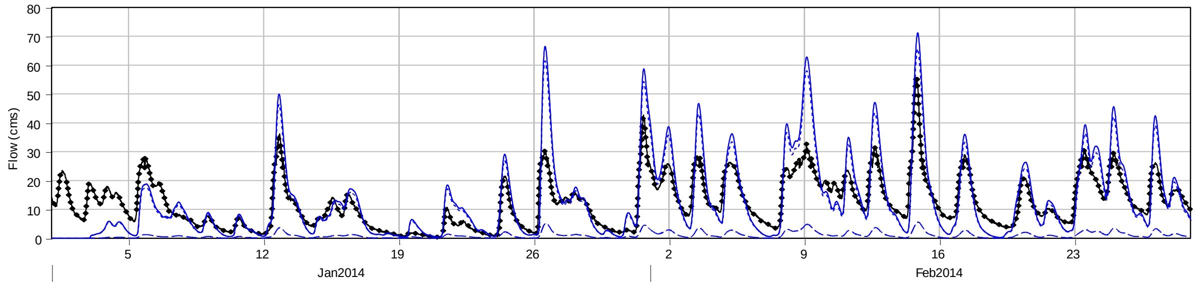
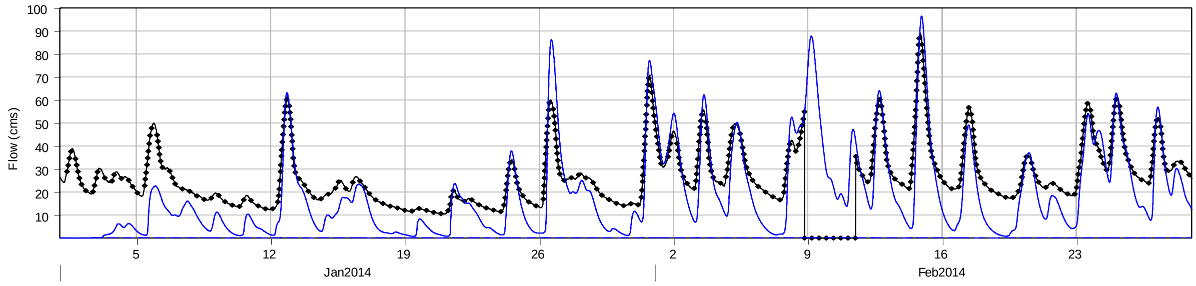


Figure 11.91. Calibration results for October 2013 (blue line - simulated; black line - observed).

a) Long Bridge – HS20008



b) Bealaboy – HS20016



c) Bandon – HS20001

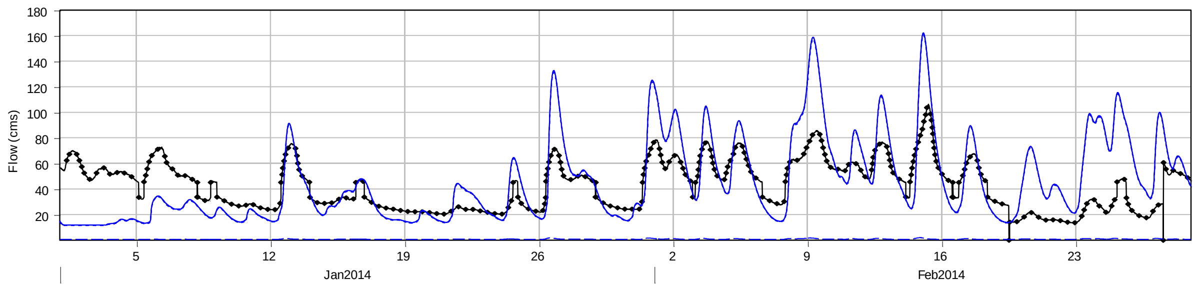
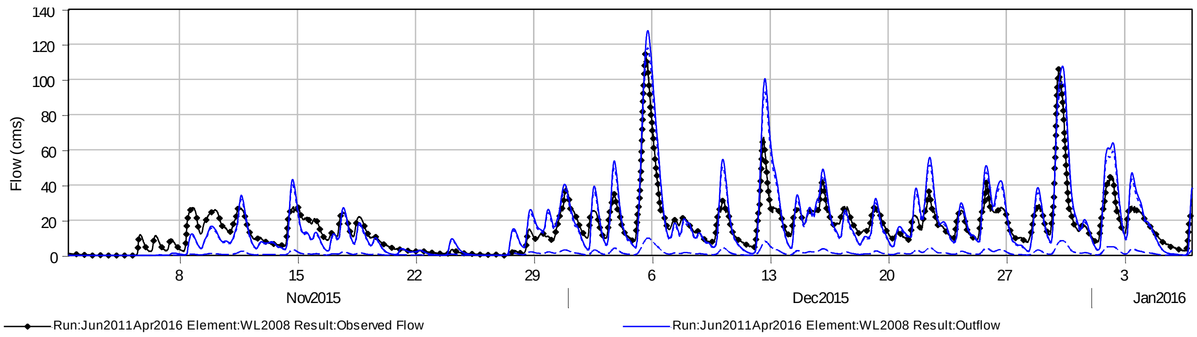
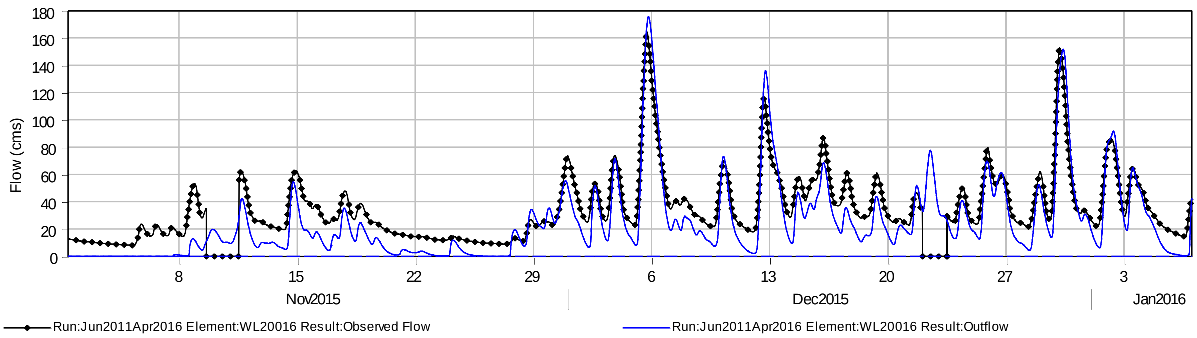


Figure 11.92. Calibration results for January and February 2014 (blue line - simulated; black line - observed).

a) Long Bridge – HS20008



b) Bealaboy – HS20016



c) Bandon – HS20001

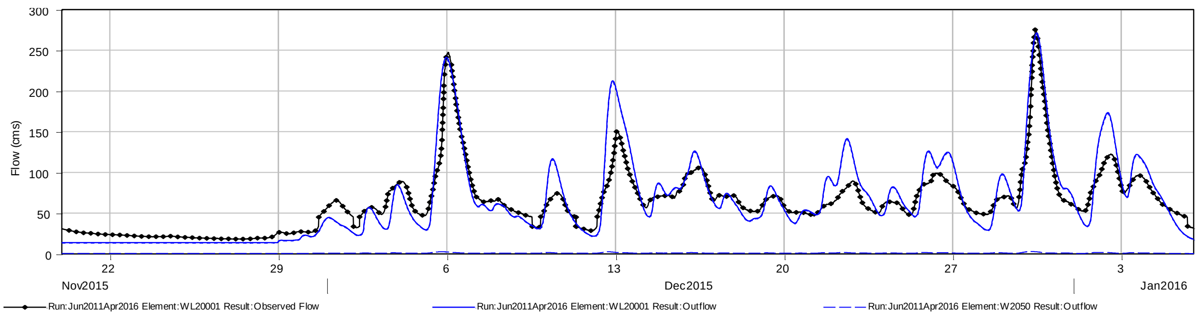
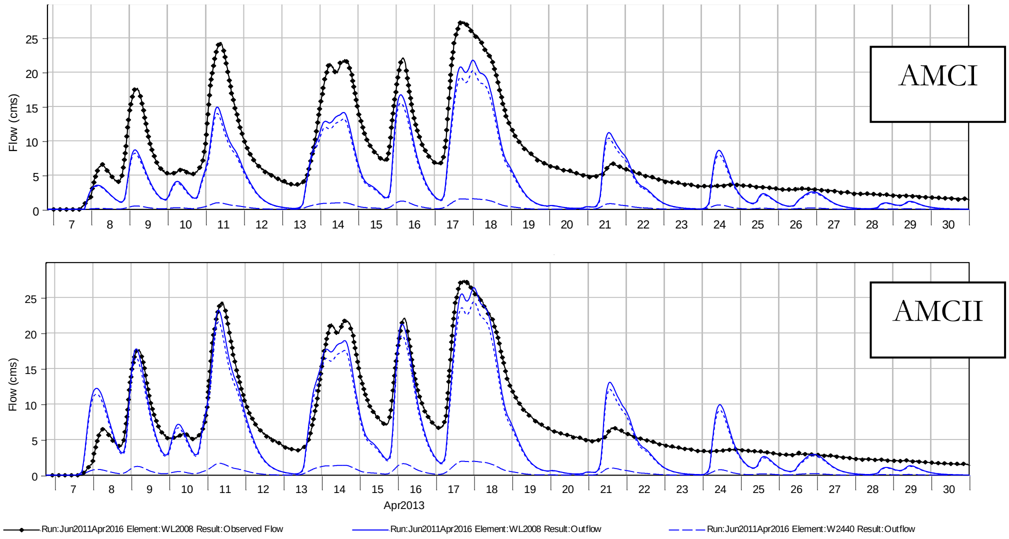
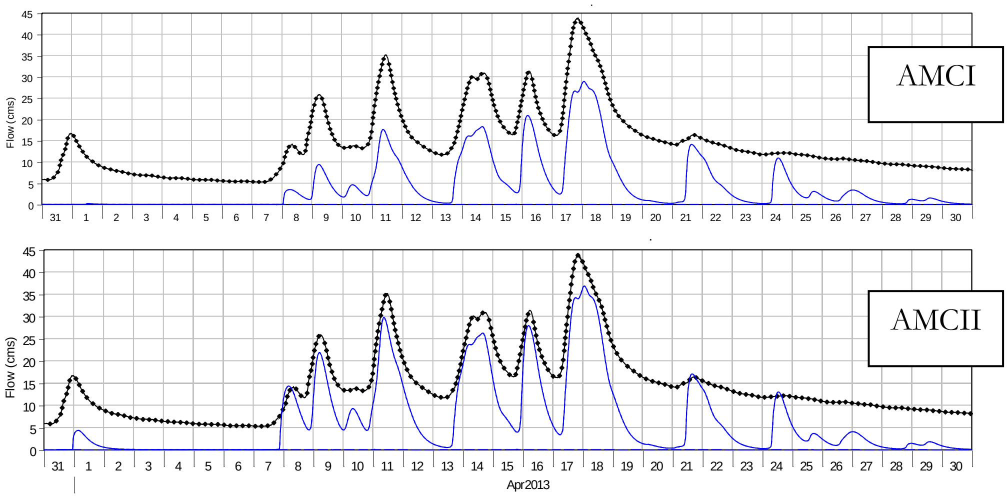


Figure 11.93. Calibration results for December 2015 (blue line - simulated; black line - observed).

a) Long Bridge – HS20008



b) Bealaboy – HS20016



c) Bandon – HS20001

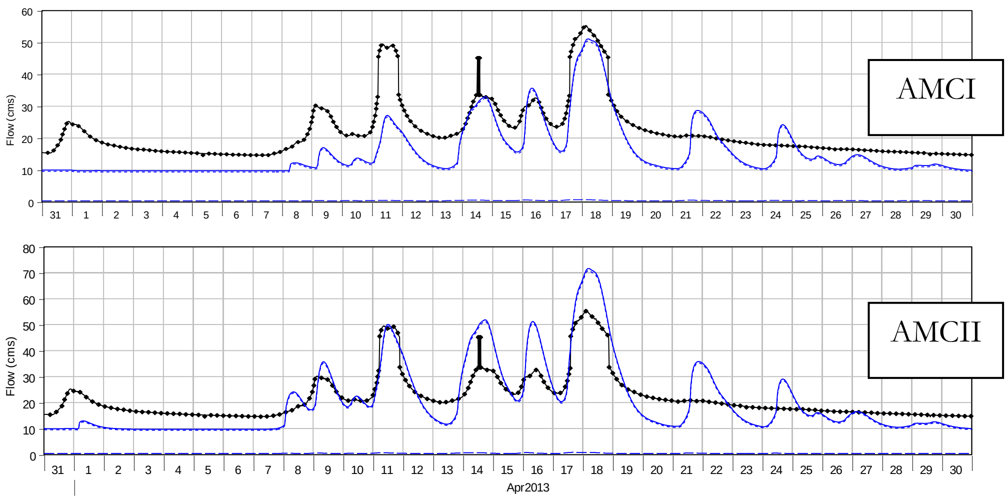


Figure 11.94. Comparison of catchment conditions for AMCI (dry) and AMCII (wet) for April 2013.

Annex O **Bridge locations and rainfall distribution**

Vermont bridge locations with corresponding damage from the rainfall from the Tropical Storm Irene in 2011 is shown below. Source: Anderson et. al [202].

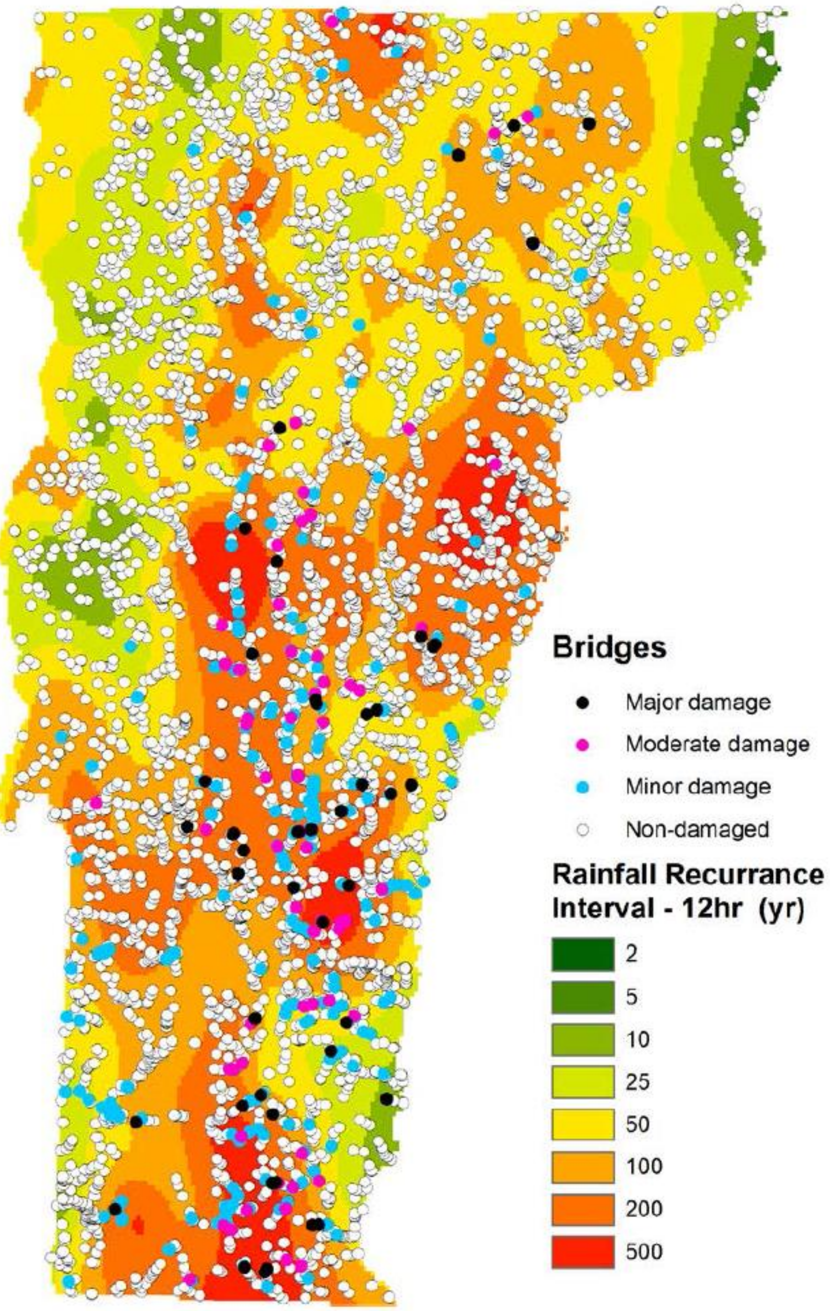


Figure 2.1: Bridge locations and rainfall from Tropical Storm Irene

Annex P Cost break down of inspection and prediction module

P.1.1 Cost of bridge inspection

The calculation is based on an average annual gross salary of €35,045.00 for Technician / Junior engineer and €54,928.00 for a Senior Engineer (source: https://ie.indeed.com/salaries) A 25% of increase of hourly wage is assumed for the overtime hours. Motor travel rate of €2.00 per kilometre is applied. A 30% margin on the overall price is applied in case that the whole bridge inspection is outsourced. All prices are without VAT.

(B1) Bekić-McKeogh Stage 1

	Bridge inspections per day: 5		Quantity	Unit cost	Total per day	Total per bridge
A	**Total man hours per inspection:**	**hours**	**3.5**	**€110.5**	**€386.60**	**€77.32**
A.1	Time of bridge inspection (hours)					
	Technician / Junior Engineer	hours	0.75	€16.85	€12.64	
	Senior Engineer	hours	0.75	€26.41	€19.81	
A.2	Time of driving to bridge					
	Technician / Junior Engineer	hours	1	€16.85	€16.85	
	Senior Engineer	hours	1	€26.41	€26.41	
A.3	Total driving + inspection					
	Technician / Junior Engineer standard rate	hours	8	€16.85	€134.79	€26.96
	Technician / Junior Engineer overtime	hours	0.75	€21.06	€15.80	€3.16
	Senior Engineer standard rate	hours	8	€26.41	€211.26	€42.25
	Senior Engineer overtime	hours	0.75	€33.01	€24.76	€4.95
B	**Total expenditure acommodation + meals + driving**				**€440.00**	**€88.00**
B.1	Cost of hotel per night	night	2	€60.00	€120.00	€24.00
B.2	Meals (lunch and dinner)	per meal	4	€20.00	€80.00	€16.00
B.3	Mileage per inspection	km	120	€2.00	€240.00	€48.00
C	**Reporting cost**		**23.00**		**€492.67**	**€98.53**
C.1	Person 1 hours for report	hours	4	€16.85	€67.39	€13.48
C.2	Person 2 hours for report	hours	8	€16.85	€134.79	€26.96
C.3	Person 3 hours per report	hours	8	€26.41	€211.26	€42.25
C.4	Person 4 hours per report	hours	2	€26.41	€52.82	€10.56
C.5	Person 5 hours per report	hours	1	€26.41	€26.41	€5.28
D	**Cost of outsourcing river bed survey**	no	**5**	**€500.0**	**€2,500.00**	**€500.00**

Subtotal	**€1,319.27**	**€263.85**
Subtotal with margin 30%	**€1,715.05**	**€343.01**
Total Man Hours	**-**	**26.50**

(B2) Bekić-McKeogh Stage 2

	Bridge inspections per day: 1		Quantity	Unit cost	Total per day	Total per bridge
A	**Total man hours per inspection:**	**hours**	**24**	**€137.4**	**€480.84**	**€480.84**
A.1	Time of bridge inspection (hours)					
	Technician / Junior Engineer	hours	14	€16.85	€235.88	
	Senior Engineer	hours	7	€26.41	€184.85	
A.2	Time of driving to bridge					
	Technician / Junior Engineer	hours	2	€16.85	€33.70	
	Senior Engineer	hours	1	€26.41	€26.41	
A.3	Total driving + inspection					
	Technician / Junior Engineer standard rate	hours	16	€16.85	€269.58	€269.58
	Technician / Junior Engineer overtime	hours	0	€21.06	€0.00	€0.00
	Senior Engineer standard rate	hours	8	€26.41	€211.26	€211.26
	Senior Engineer overtime	hours	0	€33.01	€0.00	€0.00
B	**Total expenditure acommodation + meals + driving**				**€420.00**	**€420.00**
B.1	Cost of hotel per night	night	3	€60.00	€180.00	€180.00
B.2	Meals (lunch and dinner)	per meal	6	€20.00	€120.00	€120.00
B.3	Mileage per inspection	km	60	€2.00	€120.00	€120.00
C	**Reporting cost**		**23.00**		**€492.67**	**€492.67**
C.1	Person 1 hours for report	hours	4	€16.85	€67.39	€67.39
C.2	Person 2 hours for report	hours	8	€16.85	€134.79	€134.79
C.3	Person 3 hours per report	hours	8	€26.41	€211.26	€211.26
C.4	Person 4 hours per report	hours	2	€26.41	€52.82	€52.82
C.5	Person 5 hours per report	hours	1	€26.41	€26.41	€26.41
D	**Cost of outsourcing river bed survey**	no	**1**	**€500.0**	**€500.00**	**€500.00**

Subtotal	**€1,393.51**	**€1,393.51**
Subtotal with margin 30%	**€1,811.56**	**€1,811.56**
Total Man Hours	**-**	**47**

(L1) New Level 1 inspection

	Bridge inspections per day: 5		Quantity	Unit cost	Total per day	Total per bridge
A	**Total man hours per inspection:**	**hours**	**2.5**	**€79.6**	**€278.46**	**€55.69**
A.1	Time of bridge inspection (hours)					
	Technician / Junior Engineer	hours	0.25	€16.85	€4.21	
	Senior Engineer	hours	0.25	€26.41	€6.60	
A.2	Time of driving to bridge					
	Technician / Junior Engineer	hours	1	€16.85	€16.85	
	Senior Engineer	hours	1	€26.41	€26.41	
A.3	Total driving + inspection					
	Technician / Junior Engineer standard rate	hours	5.5	€16.85	€92.67	€18.53
	Technician / Junior Engineer overtime	hours	0.75	€21.06	€15.80	€3.16
	Senior Engineer standard rate	hours	5.5	€26.41	€145.24	€29.05
	Senior Engineer overtime	hours	0.75	€33.01	€24.76	€4.95
B	**Total expenditure acommodation + meals + driving**				**€440.00**	**€88.00**
B.1	Cost of hotel per night	night	2	€60.00	€120.00	€24.00
B.2	Meals (lunch and dinner)	per meal	4	€20.00	€80.00	€16.00
B.3	Mileage per inspection	km	120	€2.00	€240.00	€48.00
C	**Reporting cost**		**0.45**		**€9.17**	**€1.83**
C.1	Person 1 hours for report	hours	0.0333	€16.85	€0.56	€0.11
C.2	Person 2 hours for report	hours	0.25	€16.85	€4.21	€0.84
C.3	Person 3 hours per report	hours	0.17	€26.41	€4.40	€0.88
C.4	Person 4 hours per report	hours	0	€26.41	€0.00	€0.00
C.5	Person 5 hours per report	hours	0	€26.41	€0.00	€0.00

Subtotal	**€727.64**	**€145.53**
Subtotal with margin 30%	**€945.93**	**€189.19**
Total Man Hours	**-**	**2.95**

(L1) New Level 2 inspection (survey part of inspection)

	Bridge inspections per day: 5		Quantity	Unit cost	Total per day	Total per bridge
A	**Total man hours per inspection:**	**hours**	**12**	**€137.4**	**€480.84**	**€240.42**
A.1	Time of bridge inspection (hours)					
	Technician / Junior Engineer	hours	14	€16.85	€235.88	
	Senior Engineer	hours	7	€26.41	€184.85	
A.2	Time of driving to bridge					
	Technician / Junior Engineer	hours	2	€16.85	€33.70	
	Senior Engineer	hours	1	€26.41	€26.41	
A.3	Total driving + inspection					
	Technician / Junior Engineer standard rate	hours	16	€16.85	€269.58	€134.79
	Technician / Junior Engineer overtime	hours	0	€21.06	€0.00	€0.00
	Senior Engineer standard rate	hours	8	€26.41	€211.26	€105.63
	Senior Engineer overtime	hours	0	€33.01	€0.00	€0.00
B	**Total expenditure acommodation + meals + driving**				**€540.00**	**€135.00**
B.1	Cost of hotel per night	night	3	€60.00	€180.00	€90.00
B.2	Meals (lunch and dinner)	per meal	6	€20.00	€120.00	€60.00
B.3	Mileage per inspection	km	120	€2.00	€240.00	€120.00
C	**Reporting cost**		**0.83**		**€22.01**	**€11.00**
C.1	Person 1 hours for report - Supervisor adding task	hours	0.083	€26.41	€2.20	€1.10
C.2	Person 2 hours for report - inspector	hours	0.5	€26.41	€13.20	€6.60
C.3	Person 3 hours per report - Supervisor closing the insp.	hours	0.25	€26.41	€6.60	€3.30
C.4	Person 4 hours per report	hours	0	€26.41	€0.00	€0.00
C.5	Person 5 hours per report	hours	0	€26.41	€0.00	€0.00

Subtotal	**€1,042.84**	**€386.42**
Subtotal with margin 30%	**€1,355.70**	**€502.35**
Total Man Hours	**-**	**12.83**

(L2) New Level 2 inspection (bathymetry survey is outsourced)

	Bridge inspections per day: 1		Quantity	Unit cost	Total per day	Total per bridge
A	**Total man hours per inspection:**	**hours**	**3**	**€95.0**	**€332.53**	**€66.51**
A.1	Time of bridge inspection (hours)					
	Technician / Junior Engineer	hours	0.5	€16.85	€8.42	
	Senior Engineer	hours	0.5	€26.41	€13.20	
A.2	Time of driving to bridge					
	Technician / Junior Engineer	hours	1	€16.85	€16.85	
	Senior Engineer	hours	1	€26.41	€26.41	
A.3	Total driving + inspection					
	Technician / Junior Engineer standard rate	hours	6.75	€16.85	€113.73	€22.75
	Technician / Junior Engineer overtime	hours	0.75	€21.06	€15.80	€3.16
	Senior Engineer standard rate	hours	6.75	€26.41	€178.25	€35.65
	Senior Engineer overtime	hours	0.75	€33.01	€24.76	€4.95
B	**Total expenditure acommodation + meals + driving**				**€540.00**	**€108.00**
B.1	Cost of hotel per night	night	3	€60.00	€180.00	€36.00
B.2	Meals (lunch and dinner)	per meal	6	€20.00	€120.00	€24.00
B.3	Mileage per inspection	km	120	€2.00	€240.00	€48.00
C	**Reporting cost**		**0.83**		**€22.01**	**€4.40**
C.1	Person 1 hours for report - Supervisor adding task	hours	0.083	€26.41	€2.20	€0.44
C.2	Person 2 hours for report - inspector	hours	0.5	€26.41	€13.20	€2.64
C.3	Person 3 hours per report - Supervisor closing the insp.	hours	0.25	€26.41	€6.60	€1.32
C.4	Person 4 hours per report	hours	0	€26.41	€0.00	€0.00
C.5	Person 5 hours per report	hours	0	€26.41	€0.00	€0.00
D	**Cost of outsourcing river bed survey**	no	**5**	**€500.0**	**€2,500.00**	**€500.00**

Subtotal	**€3,394.54**	**€678.91**
Subtotal with margin 30%	**€3,662.90**	**€732.58**
Total Man Hours	**-**	**3.83**

P.1.2 Cost of Flood Forecast and Early Warning System

(1) Required Inputs

A1	Monitoring and prediction module	Unit of Measure
1	Design Years of System Service	years
2	Estimated users for warning	Number

A2	Modelling data	
1	Is there need for hydrological model	
2	Is there need for hydraulic model	
3	Dissemination of Warning via	
	Automatic messaging system SMS (restricted to 160 characters)	24 SMS/user/year
	Email - SMPT server hosting (characters and graphics)	months
	Twitter: Free service (characters and graphics)	years
3	Enter catchment area	km^2
4	Enter number of junctions	Quantity
5	Enter km of main river	km

B	Bridge/Structure data	
	No of bridges/structures to be Monitored	Quantity
	Bridge ID	ID
	Location (optional)	String
	Easting (optional)	Decimal degrees
	Northing (optional)	Decimal degrees
	Sensors that should be instaled at the bridge	
	No of Water level sensors (contactless)	Quantity
	No of Water level sensors (presure probe)	Quantity
	No of Flowmeters	Quantity
	No of Scour (contactless - sonar) sensors	Quantity
	No of Structural Health Monitoring sensors	Quantity
	No of datalogers	Quantity

(2) Cost of gauge installation

		Unit of Measure	Quantity	Unit price	Total
A	**Sensors**				**€7,100.00**
A.1	Data logger	Quantity	1.00	€800.00	€800.00
A.2	WL Gauge (contactless - sonar)	Quantity	1.00	€300.00	€300.00
A.3	WL Gauge (pressure probe)	Quantity	1.00	€1,500.00	€1,500.00
A.4	Scour Probe	Quantity	1.00	€500.00	€500.00
A.5	Flowmeter	Quantity	1.00	€2,500.00	€2,500.00
A.6	Structural Health Monitoring	Quantity	6.00	€250.00	€1,500.00
B	**Installation sundries costs**				**€300.00**
B.1	20W Solar Power Supply to include the following, Solar panel, mounting bracket, 12V charge controller & 20Ah deep cycle battery reserve. ** Does not include cabinet/housing	Quantity	1.00	€250.00	€250.00
B.2	IP 56 enclosure to house logger, battery and solar charge controller	Quantity	1.00	€50.00	€50.00
C	**Labour**				**€539.54**
C.1	Time of bridge inspection (hours)				
	Technician / Junior Engineer	hours	4	€16.85	€67.39
	Senior Engineer	hours	4	€26.41	€105.63
C.2	Time of driving to bridge				
	Technician / Junior Engineer	hours	2	€16.85	€33.70
	Senior Engineer	hours	2	€26.41	€52.82
C.3	Meals (lunch and dinner)	per meal	2	€20.00	€40.00
C.4	Mileage per inspection	km	120	€2.00	€240.00
D	**Annual Maintenance costs**				**€157.70**
D.1	*Technician / Junior Engineer*	hours	2	€16.85	€33.70
D.2	Mileage per inspection	km	60	€2.00	€120.00
*	*Salaries*	*Annual*	*per hour*		
	Technician / Junior Engineer	*€35,045.00*	*€16.85*		
	Senior Engineer	*€54,928.00*	*€26.41*		

(3) FEWS Development and running costs

No	Description of work / service	Unit of Measure	Quantity	Unit Price	Total Price
A	**Initial Setup of monitoring and prediction module (Hydrology Modelling)**			**Yes**	**€10,500.00**
1	**Site Measurements - water levels and flow velocities**	days	3.00	€1,000.00	€3,000.00
2	**Hydrological analysis**				
	Obtaining historic data (rainfall, water level, flow rate) and rating curves	days	2.00	€500.00	€1,000.00
	Rainfall-Runoff for duration 1H, 3H, 6H, 12H 24H.	days	2.00	€500.00	€1,000.00
	Tides	days	1.00	€500.00	€500.00
2	**Hydrology model development**				
	Preparing DTM, shapefiles (River, Catchment) 2 days	days	2.00	€500.00	€1,000.00
	Preparing Land Cover and Soil Maps for Ilen Catchment 1 day	days	1.00	€500.00	€500.00
	Calculation of Curve Number 2 days	days	2.00	€500.00	€1,000.00
	Calculation of Catchment characteristics (time of concentration, slopes, etc.) 1 day	days	1.00	€500.00	€500.00
	Calculation of River Lengths and slopes 1 day	days	1.00	€500.00	€500.00
	Catchment delineation and creating of the HEC-HMS model 3 days	days	3.00	€500.00	€1,500.00
B	**Initial Setup of monitoring and prediction module Hydraulic Modelling)**			**No**	**€6,300.00**
	Gathering the existing geometry and additional topographical Survey of river channel, structures, floodplains and ponds.	days	5.00	€500.00	€5,400.00
	Development of 1D hydraulic model HEC RAS.	days	3.00	€150.00	€450.00
	Calibration and verification of HEC-RAS model	days	3.00	€150.00	€450.00
A+B	**Modelling Overall price**				**€10,500.00**
C	**Flood Forecast**				**€10,500.00**
	Setting up the pilot area and locations	days	1.00	€1,500.00	€1,500.00
	Automation of Imports from observations and rainfall forecasts	days	1.00	€1,500.00	€1,500.00
	Automation of Hydrologic model	days	2.00	€1,500.00	€3,000.00
	Automation of Hydraulic model	days	2.00	€1,500.00	€3,000.00
	Defining exports and reporting	days	1.00	€1,500.00	€1,500.00
D	**Flood Warning setup**				**€6,750.00**
	Plotting of recorded and rainfall forecast (table and graph form)	days	0.50	€1,500.00	€750.00
	Definition of locations for warning and thresholds	days	0.50	€1,500.00	€750.00
	Defining outputs for warning	days	0.50	€1,500.00	€750.00
	Flow hydrograph output set-up.	days	1.00	€1,500.00	€1,500.00
	Tide information output set-up.	days	1.00	€1,500.00	€1,500.00
	Development of ICT warning service (SMS, email, twitter or Telegraph)	days	1.00	€1,500.00	€1,500.00
7	**Issue of Warnings (choose from below)**				
	Automatic messaging system (restricted to 160 characters)	per text	20,000.00	€0.05	€1,000.00
	Email - smpt server hosting (characters and graphics)	months	12.00	€70.00	€840.00
	Twitter: Free service (characters and graphics)	annual	1.00	€0.00	€0.00

8	**Project Management**	days	3.00	€1,250.00	€3,750.00
E	**Running costs per year**	year	**2.00**	€830.00	€1,660.00
	Site visit for monitoring equipment every 6 months	days	4.00	€500.00	€2,000.00
	Licences				
	Licence 1	Quantity			
	…	Quantity			
	Licence 2	Quantity			
	Buying forecast for tides	annually	1.00	€50.00	€50.00
	Hosting service (Server 120GB HDD, 32GB RAM)	months	12.00	€280.00	€3,360.00
	Running servers for hosting Delft FEWS and computations				

Total (VAT not included):	**€12,160.00**
VAT 23%:	**€2,796.80**
SUBTOTAL :	**€14,956.80**

(4) Output: Estimated cost(s) of FFS and FEWS system

The costs below are estimated for the 600m^2 catchment area, assuming installation of

		Unit of Measure	Quantity	Unit price	Total
C	**Costs**				**€67,049.83**
C.1	Cost of modelling				**€10,500.00**
C.2	Installation of Monitoring equipment				**€28,274.91**
	Minimum No of Rain gauges	Quantity	3	€2,439.54	€7,318.61
	Minimum No of Water level Gauges	Quantity	4	€1,939.54	€7,758.15
	Installed monitoring at the bridge	Quantity	4	-	€13,198.15
C.2	Cost of Flood Forecast				**€10,500.00**
C.3	Cost of Flood Warning				**€6,750.00**
C.4	Running cost of FEWS	Years	2	€1,660.00	**€3,320.00**

** Recommended minimum requirements*

1 Rain gauge per	*200 km^2*
2 Water level gauges per junction	*2 per junction*
3 Water level gauges per km	*25 km*